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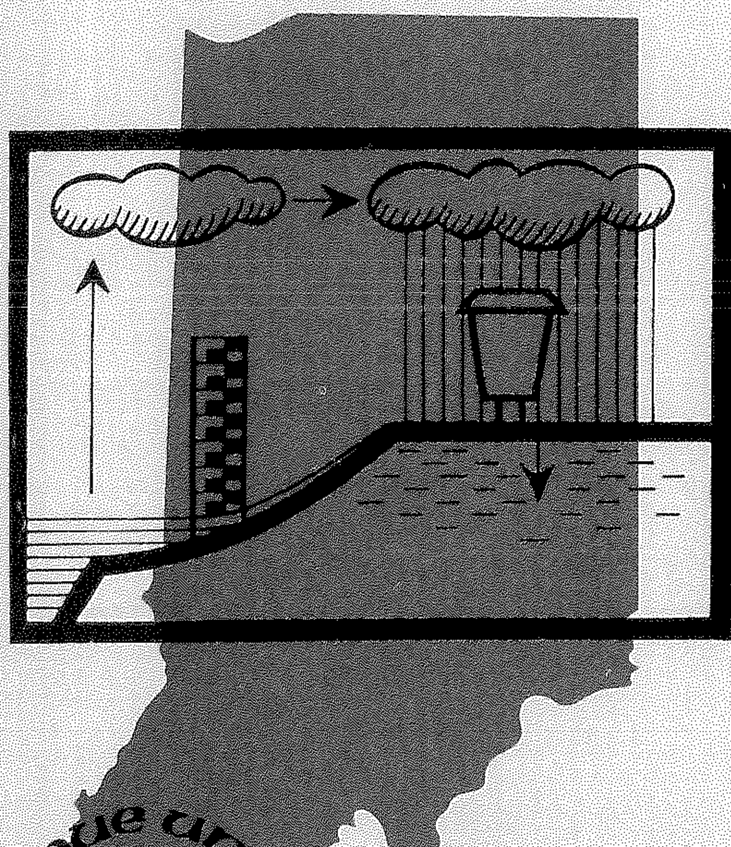

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A. Ramachandra Rao

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September 1980

PURDUE UNIVERSITY
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ABSTRACT

The effort which has been expended in the past two decades on the development of urban hydrologic models is quite significant. However, a considerable amount of work still needs to be done on these models in several areas. The present study deals with one of these aspects, namely the least cost design of urban drainage systems by using urban rainfall-runoff models.

The importance of hydrlogic models in planning, design and operation of urban drainage systems has been repeatedly mentioned and is generally accepted. However, application and use of such models to explore alternative designs and their associated costs has not been widely reported. Several of the least cost design models assume that the inflow rates are accurately known. Without a good hydrologic model, these inflow rates cannot be accurately estimated and hence the least cost design model may not be used with confidence.

In the present study, a simple, readily usable, and theoretically sound model which includes ILLUDAS and a dynamic programming subroutine is developed. The model is tested by using data from Upper Ross-Ade and Bar Barry Heights watersheds in West Lafayette, Indiana. The model is used to investigate the variation of system costs with the variation in design parameters and to optimally size the detention storage volume for the least cost of the system.

CHAPTER I

INTRODUCTION

Urban communities are often faced with problems related to stormwater management. Usually a system collecting and transporting surface runoff from storms and which costs the least is sought. The least cost system is not the only concern to most communities, however. They are equally interested in other sociological effects such as the employment generation on their communities by the construction of a drainage system (Miller, (1978)). However, the cost of the drainage system is still the main issue when drainage systems are planned, designed, and constructed.

Traditionally, little attention is explicitly paid in the design process to the cost of drainage systems. Designers usually follow design manuals, guidelines, text books, and often "engineering judgment" to design drainage systems. Whether the design adopted is an optimal one, especially whether it is the least cost one, is usually not explored. One can very easily verify that a larger pipe can carry the same amount of runoff as a small one which is laid on a steeper slope. Such tradeoffs exist i should be considered in the design of drainage systems.

Furthermore, advantages of including detention storage basins in drainage systems has been investigated and including them has been found to be generally favorable (Poertner (1973), Kamedulski and McCuen (1979)). Detention storage basins are designed to overcome the problems of peak flows, sediment loads, and pollution due to urbanization. Smaller peak flows allow smaller downstream pipe sizes and result in reduced pipe network costs.

Costs of urban drainage systems have received the attention of planners, designers, and decision makers. Although the drainage system layout and topographical information are known beforehand, usually there is a large number of feasible drainage networks which may be designed. The number of feasible drainage networks is restricted by the physical and legal constraints. These are, for instance, the maximum and minimum permissible velocities in sewers, the minimum depth required to lay the pipe, and the commercially available pipe sizes, etc. The main problems therefore, are to determine the dimensions of structures such as detention storage basins, the depths at which pipes are to be laid, and the pipe dimaeters required to remove stormwater so as to meet the design requirements at minimum costs.

Several studies regarding urban drainage system costs have been published. The regression analysis of the costs of drainage systems and the design storm and other watershed parameters has been developed by Rawls and Knapp (1972) and by Rawls and McCuen (1978). Grigg and O'Hearn (1976) developed relationships between total costs and design parameters for an urbanizing watershed in Colorado. The rational formula was used in their investigation. Mays and Yen (1975) developed methodologies for optimum least cost design of sewer systems by using dynamic and discrete differential dynamic programming algorithms. Their approaches were tested on a hypothetical sewer system in which the layout and the design flow rates were known. Mays and Wenzel (1976) also developed a model using the discrete differential dynamic programming algorithm. The design flow rates into the sewer system and the physical characteristics of the system are assumed to be known by Mays and Wenzel (1976). Froise and Burges (1978) developed a model incorporating the dynamic programming technique to determine the least cost design of the urban drainage system. Their hydraulic design subroutine is based on the Darcy-Weisbach formula. Although these least cost design models are quite useful, design flow rates are either assumed or roughly estimated in all these models. It is not possible to use these optimal design models with confidence if the design flow rates are not properly estimated. Combining the optimal least cost models with a good hydrologic model would be more useful to design engineers.

The importance of hydrologic models in planning, design, and operation of urban drainage systems have been repeatedly mentioned. However, the application and the use of such models to explore alternatives and their associated costs have not been widely reported. Consequently, the major objective of the study reported herein is to develop a simple, theoretically sound, and practically usable model to develop least cost urban drainage systems. The model which has been developed in the present study consists of an urban drainage design model and the dynamic programming algorithm. ILLUDAS is the drainage design model used in this study. This model can be used to determine the least cost urban drainage system design subject to constraints; to investigate the relationships between costs and the design parameters; and to study the variability in drainage costs with and without detention storage basins. The specific objectives of the study are the following:

(1) To develop a method so that least cost drainage designs may be attained by considering design parameters, the design storm duration and frequency, the antecedent moisture condition, and the rainfall temporal distribution; and

(2) To investigate the variability in drainage system costs with and without detention storage ponds.

The least cost design model has two major components. These are, (1) the hydrologic, and (2) the dynamic programming submodels. The hydrologic submodel used in this study is the ILLUDAS model. The hydrologic submodel is used to calculate the runoff at various points in the watershed by using the input data. These calculated runoff values are then used as inputs to the dynamic programming subroutine which gives the needed least cost design . The general structure of the least cost design model is presented in Fig. 1.1.

The report is organized as follows. The data used in the study are discussed in Chapter II. A discussion of ILLUDAS is found in Chapter III. The dynamic programming algorithm is discussed in Chapter IV. The results from the least cost model are given in chapter V. The results are discussed and a set of conclusions presented in Chapter VI. The computer program used in the study is given in the appendix.

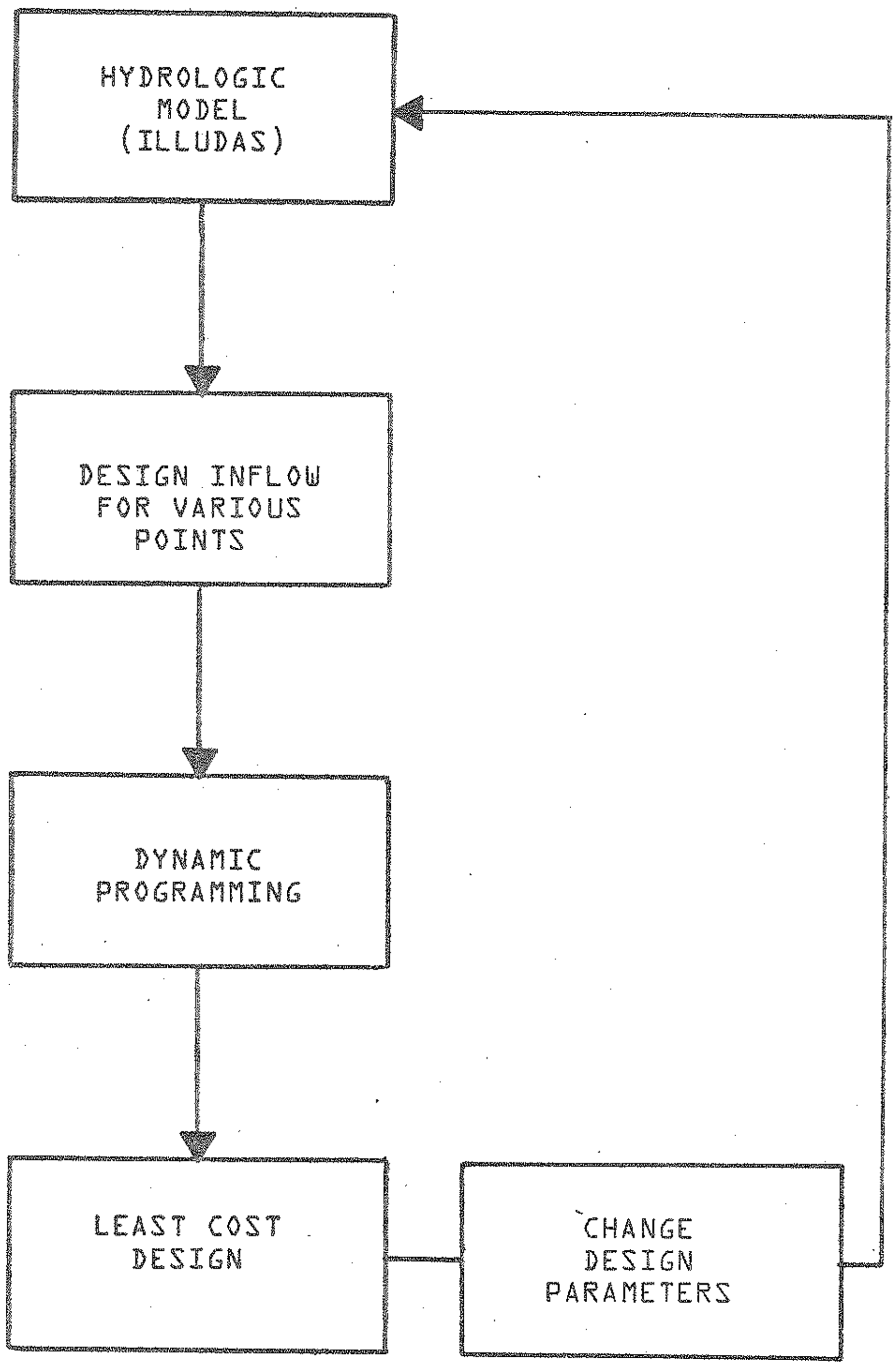

Figure 1.1 The Least Cost Design Model Structures

CHAPTER II

DATA USED IN THE STUDY

2.1 Introduction

Data from two watersheds, the Upper Ross-Ade Watershed and the Bar Barry Heights Subdivision in West Lafayette, Indiana were used in this study. A description of these watersheds, a discussion of the costs of urban drainage system components and of the data used in the study with the ILLUDAS model follows.

2.2 Upper Ross-Ade Watershed

The Upper Ross-Ade Watershed is located in West Lafayette, Indiana. Its location is shown in Fig. 2.1 and some of the details of the watershed are given in Fig. 2.2. The Upper Ross-Ade Watershed has an area of 29 acres. It is mainly residential and is relatively uniform in character. The watershed extends in a generally north-south direction and about 38 percent of its area is impervious to infiltration (Vician and Delleur (1966)). The soil types in the watershed range from Crosby silt loam in the lower levels to Miami silt loam on the steeper portions to Eel

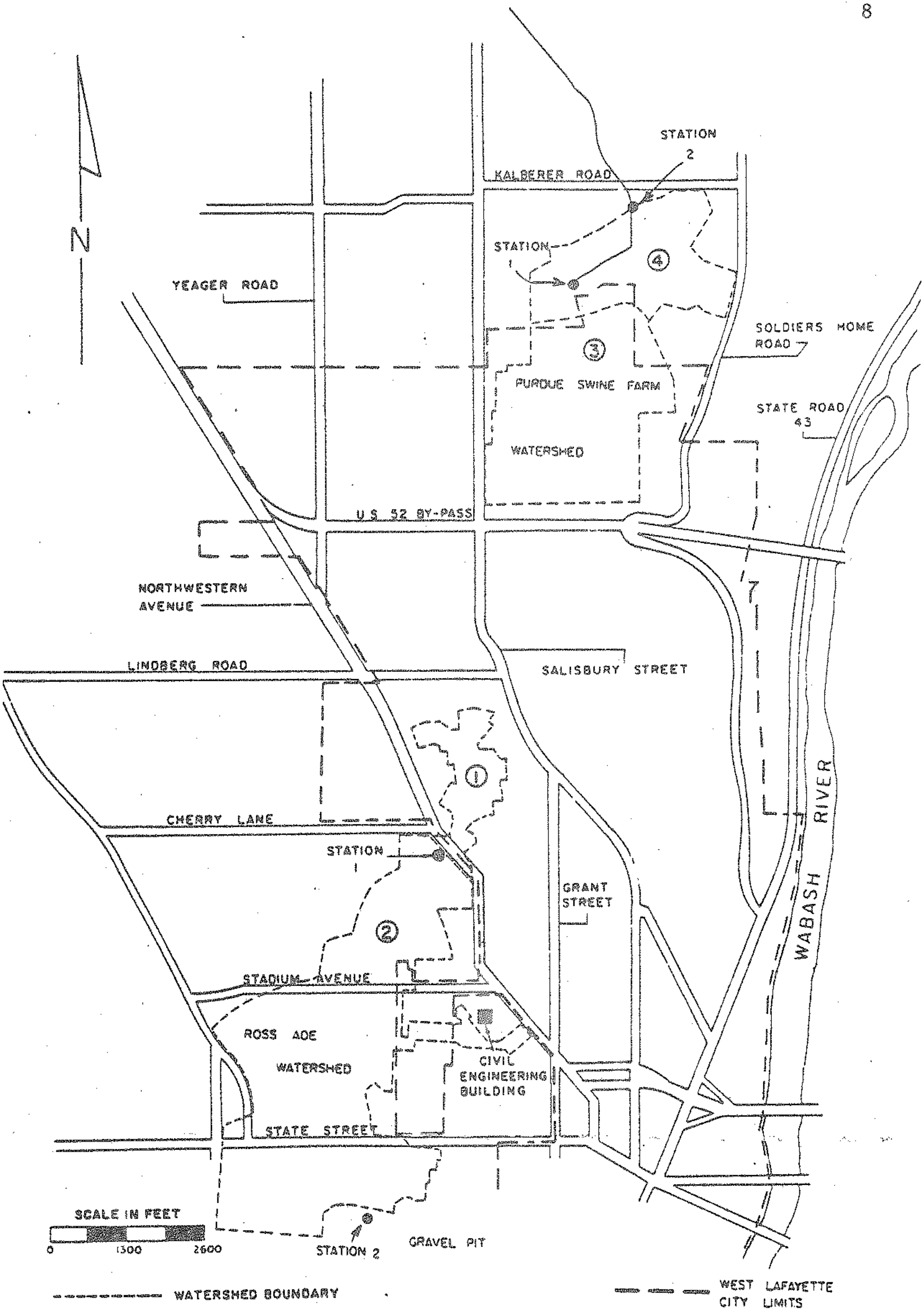

Figure 2.1 Location of the Upper Ross-Ade Watershed (From Sarma (1970))

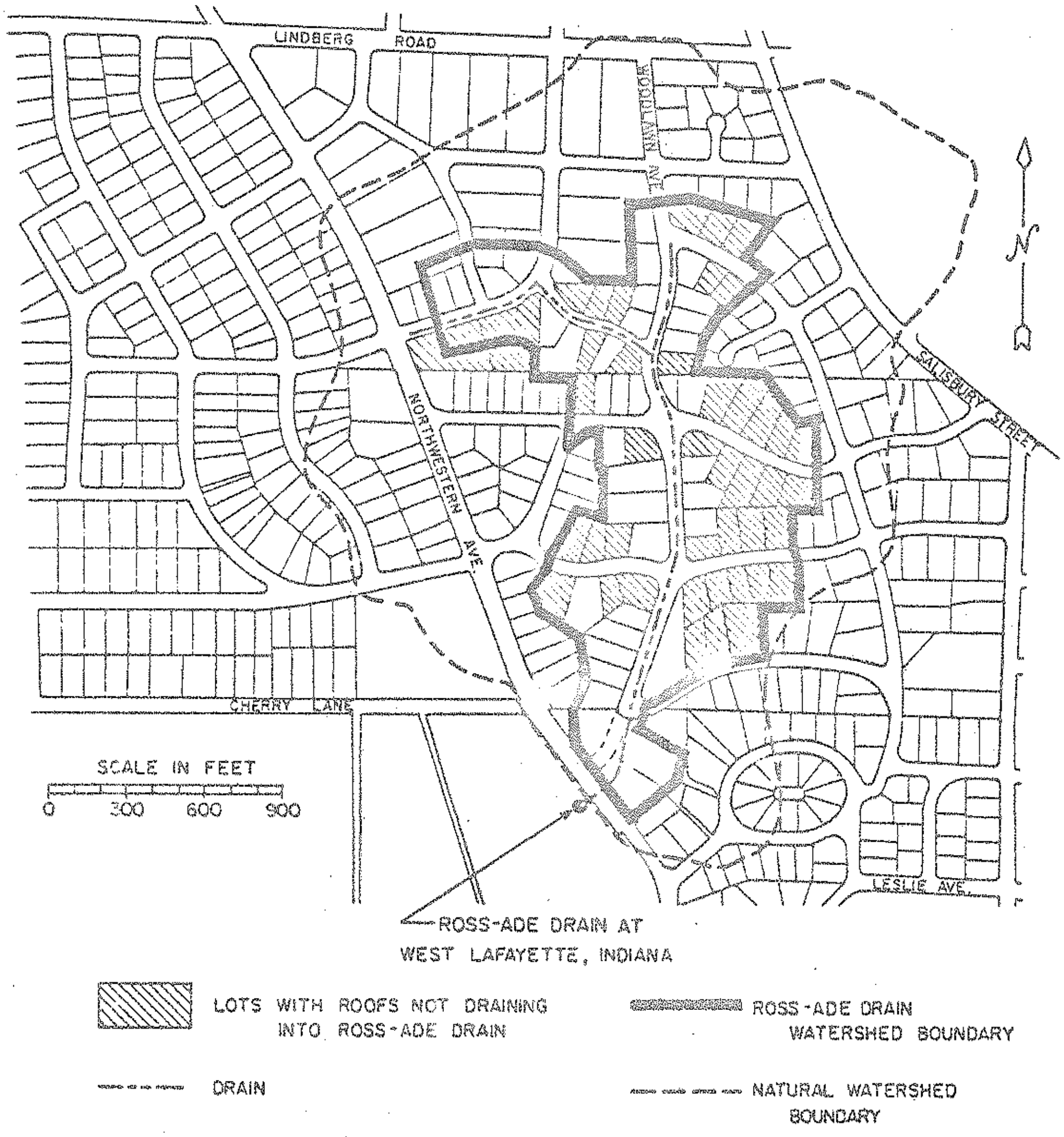

Figure 2.2 Upper Ross-Ade Watershed (From Han and Delleur (1979))

silt loam on the uplands as shown in Fig. 2.´ nd summarized in Table 2.1 (Hossain et al. (1974)). Details of the gaging station, instrumentation, data acquisition, and physiographic characteristics of the watershed have been described by Sarma (1970).

For evaluating an existing system, the physical layout as well as various details of the system are needed. These are usually obtained from a set of detailed drainage system plan and profile drawings. Such details for the Upper Ross-Ade watershed are obtained from the Office of the City Engineer, West Lafayette, Indiana. A schematic representation of the drainage system is shown in Fig. 2.4. The length, slope, and diameter of each pipe segment are listed in Table 2.2.

2.3 Bar Barry Heights Subdivision

Bar Barry Heights Subdivision is located in West Lafayette, Indiana. Its plan drawing is shown in Fig. 2.5. The drainage subbasin and storm sewer layout are shown in Fig. 2.6 (Burke (1979)). Bar Barry Heights is a middle class residential area of 121.9 acres. It is generally flat and consists of about 30 percent impervious surface area. The soil type in the watershed is classified as SCS Type B. The pipe lengths, ground elevations, slopes and other relevant topographical information were determined by Burke (1979).

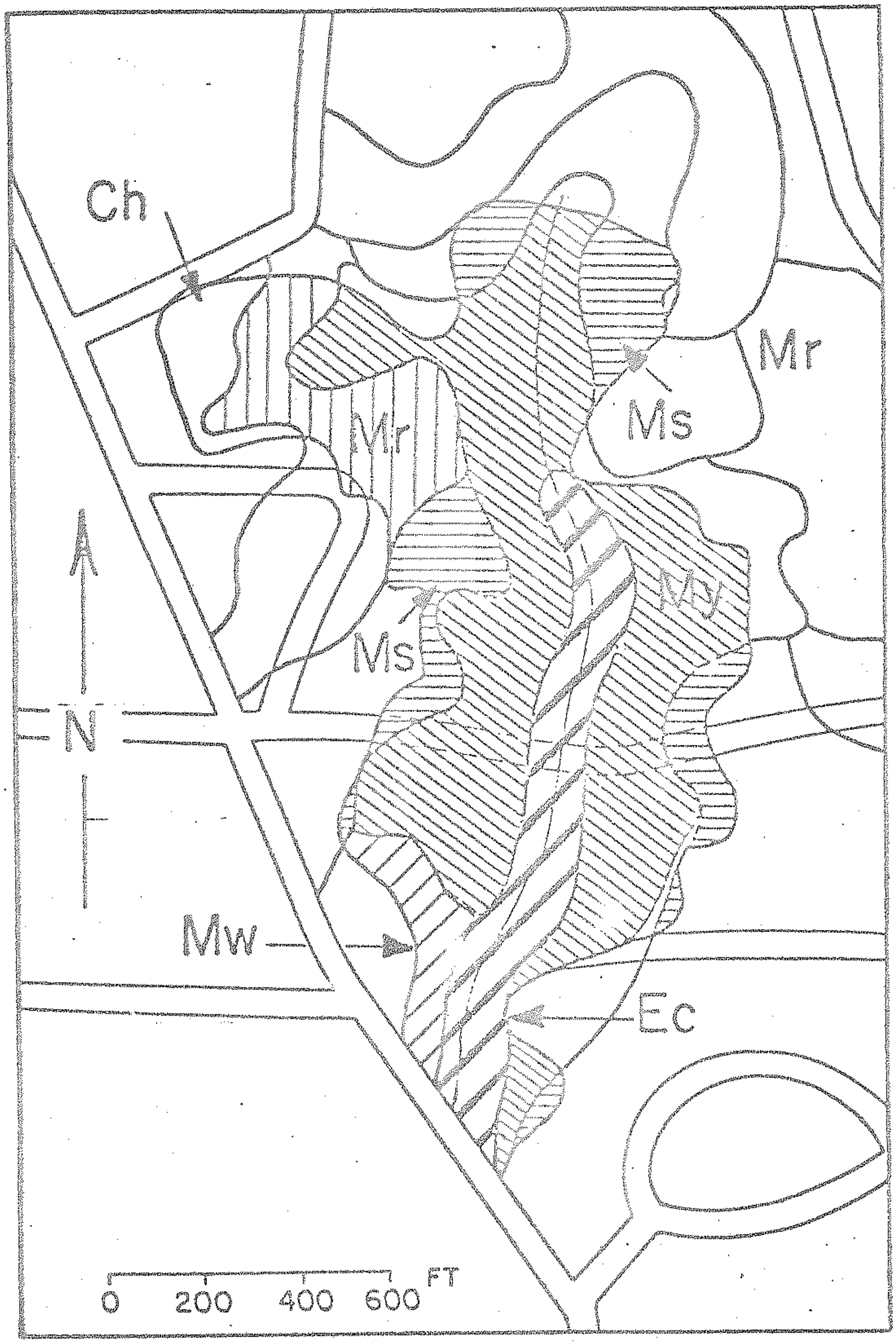

Figure 2.3 Soil Map-Upper Ross-Ade Watershed (From Hossain, et al. (1974))

Table 2.1 Soil Types in Upper RossAde Watershed (From Hossain, et al. (1974))

Symbol	Soil Type	Depth of Top Layer (in)	Area (%)
Ec	Eel silt loam 03% slope	6.0 to 8.0	16
Ms	Miami silt loam 38% slope	3.0 to 8.0	13
Mr	Miami silt loam 38% slope	7.0	12
Ch	Crosby silt loam 03% slope	7.0	5
Mw	Miami silt loam 1225% slope	3.0 to 8.0	5
Mv	Miami silt loam 1225% slope	8.0 to 12.0	49

Table 2.2 Physical Characteristics of Pipe Segments in the Upper Ross-Ade Watershed (From Han and Delleur (1979))

Branch	Reach	Length (ft)	Slope (%)	Diameter (in)
1	0	240	3.4	18
1	1	260	3.4	21
2	0	250	4.8	15
2	1	280	4.0	18
1	2	240	2.0	30
1	3	536	2.8	36
1	4	860	1.5	36

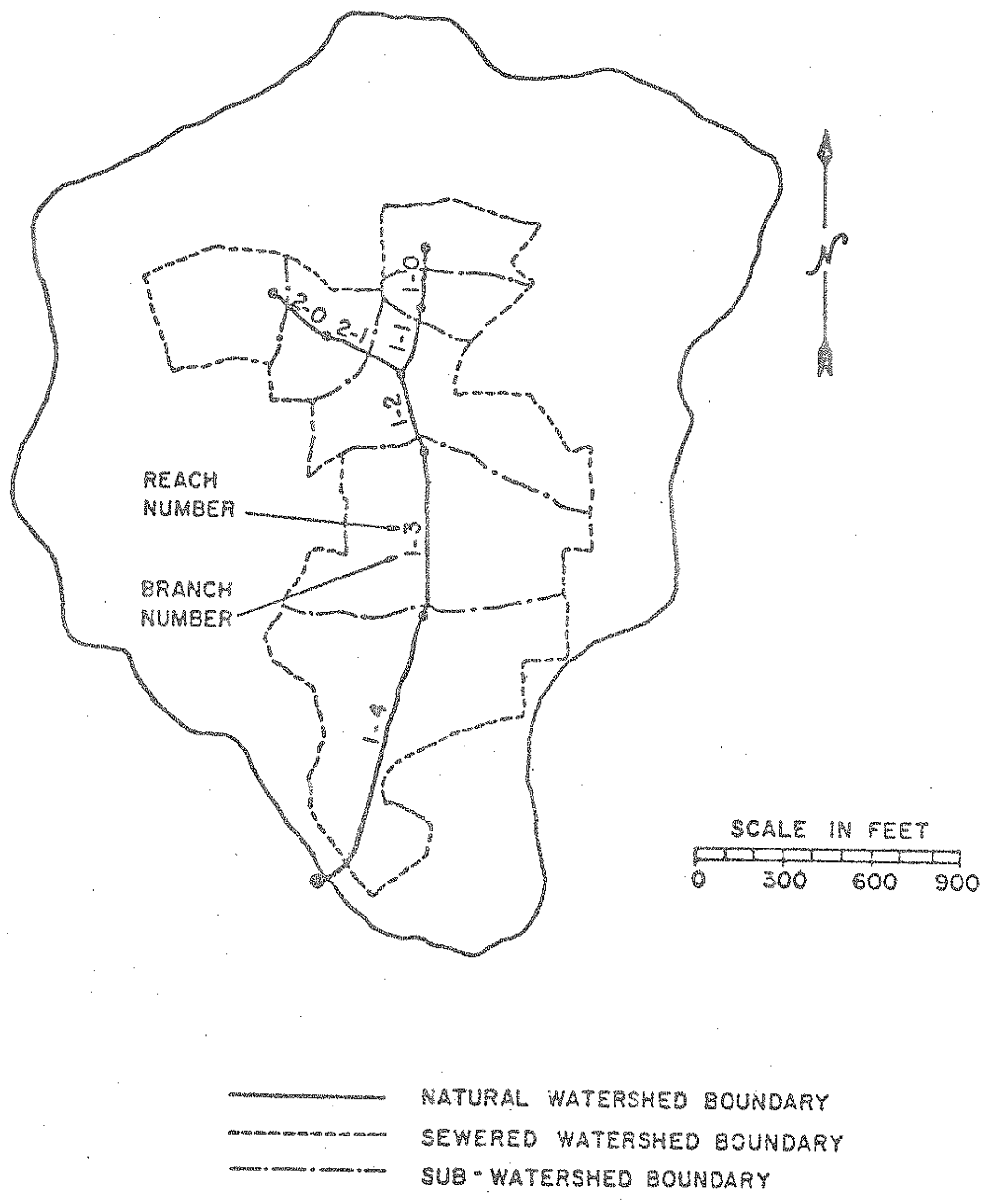

Figure 2.4 Schematic Representation of the Drainage System in Upper Ross-Ade Watershed

Figure 2.5 Plan Drawing of the Bar Barry Heights Subdivision
(From Burke (1979))

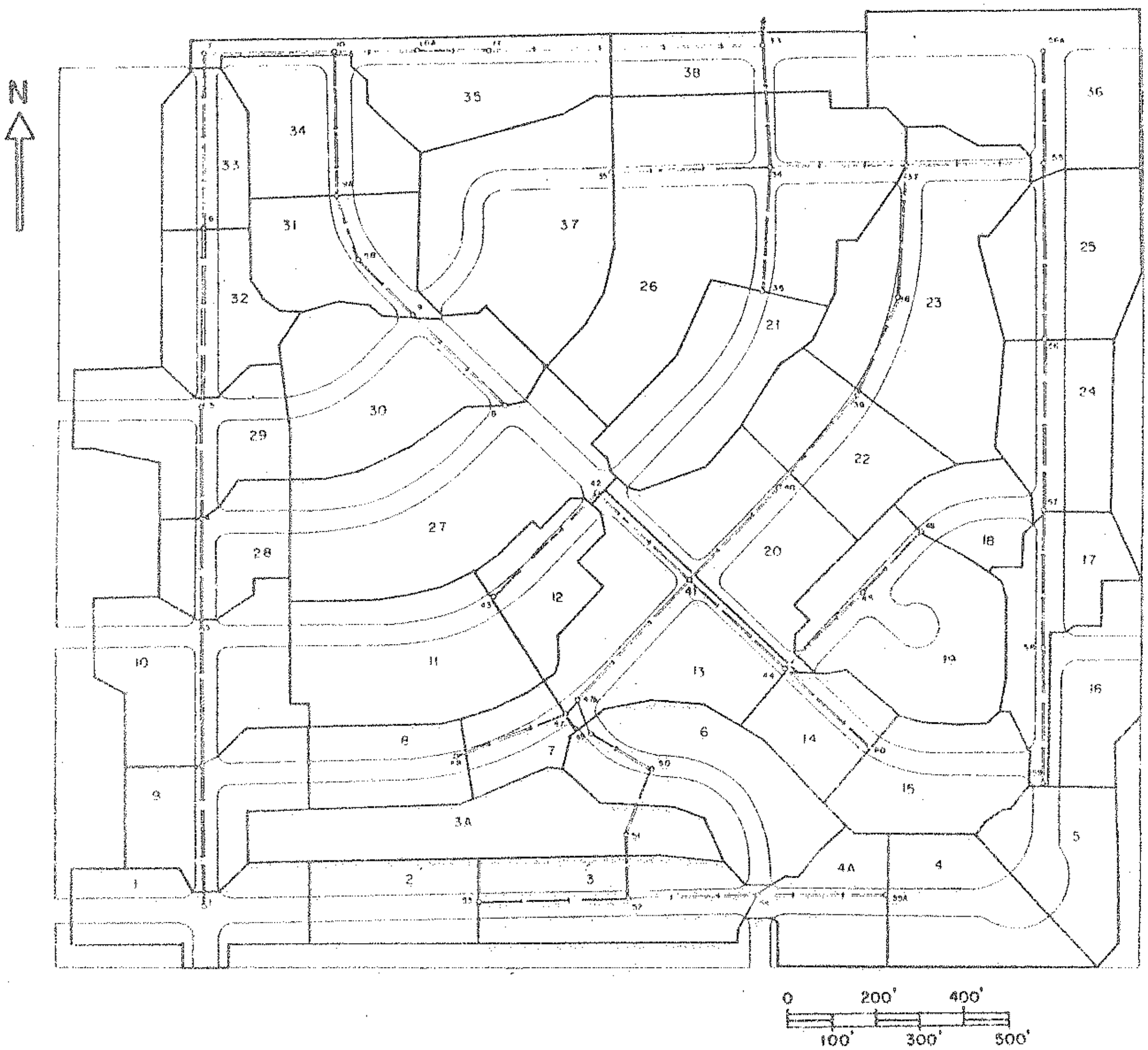

Figure 2.6 Drainage Subbasins and Storm Sewer Layout of Bar Barry Heights Subdivision (From Burke (1979))

The data from this subdivision are used in developing the optimal least cost drainage system design model. Bar Barry Heights Subdivision is divided into 40 subbasins served by 44 pipes. The detailed information for the subbasin characteristics which are found in Burke (1979) are reproduced in Table 2.3. Data from this subdivision are also used to investigate the relationships between the costs of drainage systems and some of the important urban runoff model parameters which govern runoff from this watershed.

2.4 Cost Data

The unit costs of materials were obtained from local manufacturers and from several published sources (Zoller and Rolf (1977), Miller (1978), Godfrey (1979)). These were used in the cost analysis in Chapter V. These unit costs are updated to 1980 values by using the Engineering News-Record cost indexes and are listed in Table 2.4.

The cost equations were derived starting from a literature review. It was found that many investigators had used different cost functions in their studies. Because of the locality and time differences between the studies, these cost functions are not identical. Deb and Sarkar (1971) developed a nonlinear cost equation including initial cost, transportation, laying and profit as a function of pipe diameter. Walsh and Brown (1973) used arrays of unit costs for different excavation conditions, pipes, manholes, and

Table 2.3 Subbasin Characteristics of Bar Barry Heights Subdivision (From Burke (1979))

Sub-Basin No.	MH_U-MH_D	Branch-Reach	Basin Area (Acres)	DCPA (Acres)	SPA (Acres)	GA (Acres)	Paved Length (ft)	Paved Slope (%)	Grassed Length (ft)	Grassed Slope (%)
4	55A-54	1-0	2.55	0.64	0.23	1.68	400	0.50	100	2.00
4a	54-52	1-1	1.93	0.45	0.13	1.35	330	0.40	110	2.00
3	52-51	1-2	3.00	0.78	0.19	2.03	320	0.77	70	2.00
3a	51-50	1-3	3.79	0.00	0.46	3.33	0.00	0.00	950	2.00
6	50-49	1-4	3.75	0.94	0.35	2.46	275	1.1	125	1.00
6	49-47A	1-4	--	--	--	--	--	--	--	--
7	47A-41	1-5	1.14	0.40	0.05	0.69	270	0.62	71	2.00
13	41-40	1-6	3.88	0.94	0.35	2.59	410	0.70	70	2.00
20	40-39	1-7	3.05	0.76	0.25	2.04	580	0.78	100	2.00
22	39-38	1-8	2.59	0.38	0.20	2.01	250	0.50	190	2.00
-	38-37	1-9	--	--	--	--	--	--	--	--
23	37-34	6-5	5.99	1.13	0.46	4.4	550	0.40	300	2.00
26	34-33	12-1	8.61	1.43	0.65	6.53	350	0.30	500	1.00
38	33-out	12-2	2.13	0.61	0.16	1.36	350	0.60	65	2.00
1	1-2	9-0	2.55	0.78	0.07	1.76	300	1.00	70	2.0
9	2-3	9-1	2.66	0.73	0.14	1.79	280	0.30	150	2.0
10	3-4	9-2	3.74	1.08	0.30	2.36	290	0.33	169	2.0
28	4-5	9-3	1.84	0.61	0.10	1.13	240	1.00	60	2.0
29	5-6	9-4	3.08	0.90	0.19	1.99	240	0.95	90	2.0
32	6-7	9-5	2.00	0.67	0.10	1.23	400	0.35	100	2.0
33	7-10	9-6	1.45	0.60	0.10	0.75	380	0.40	80	2.0
34	10-10A	9-7	2.78	0.66	0.23	1.89	300	0.75	200	2.0
35	10A-11	9-8	3.44	0.74	0.18	2.52	550	0.60	100	2.00
-	11-33	9-9	--	--	--	--	--	--	--	--
27	8-9	10-0	5.81	1.25	0.41	4.15	650	0.34	150	2.0
30	9-98	10-1	4.15	0.75	0.30	3.10	290	0.70	120	2.0
-	9B-9A	10-2	--	--	--	--	--	--	--	--
31	9A-10	10-3	2.59	0.51	0.21	1.87	220	0.75	100	2.0
5	59-58	6-0	1.42	0.52	0.140	0.85	320	0.25	100	2.00
16	58-57	6-1	1.93	0.47	0.160	1.30	350	0.45	150	2.00
17	57-56	6-2	2.23	0.65	0.120	1.46	600	0.65	100	2.00
24	56-55	6-3	2.41	0.65	0.08	1.68	410	0.95	95	2.00
25	55-37	6-4	3.15	0.61	0.18	2.36	400	0.70	150	2.0
18	46-45	4-0	1.35	0.42	0.15	0.78	300	0.30	100	2.00
19	45-44	4-1	4.18	0.84	0.20	3.14	170	1.00	135	2.00
14	44-41	4-2	1.57	0.36	0.10	1.11	260	0.90	144	2.00
15	60-44	5-0	2.46	0.58	0.16	1.72	390	0.65	100	2.00
11	43-42	8-0	3.80	0.78	0.36	2.66	520	0.41	180	2.00
12	42-41	8-1	1.60	0.44	0.12	1.04	330	0.81	175	2.00
37	35-34	11-0	5.39	0.93	0.26	4.20	700	0.50	300	1.00
21	36-34	12-0	3.03	0.88	0.13	2.02	580	0.95	100	2.00
2	53-52	2-0	1.93	0.53	0.08	1.33	410	0.97	70	2.00
36	56A-55	7-0	5.20	1.47	0.31	3.42	260	1.00	190	2.00
8	48-47	3-0	1.77	0.60	0.07	1.10	380	1.10	68	2.00
-	47-47A	3-1	--	--	--	--	--	--	--	--

Table 2.4 Material Unit Costs

Pipe	Diameter (in)	Pipe Cost ($/linear foot)	
	12	5.25	
	15	7.00	
	18	8.70	
	21	11.35	
	24	14.45	
	27	17.45	
	30	20.70	
	33	26.00	
	36	31.10	
	42	41.10	
	48	49.95	
	54	61.40	
	60	78.80	
	66	96.40	
	72	116.80	
	78	130.00	
Excavation	Volume (Cu. Yards)	Cost ($/Cu. Yard)	
	0-1,999	4.10	
	2,000-4,999	5.75	
	5,000-9,999	3.34	
	10,000-24,999	2.75	
	25,000-49,999	3.30	
	50,000-99,999	2.15	
	100,000 and Over	1.51	
Concrete Surface	Surface Area (Sq. Yards)	Wall Thickness (in)	Cost ($/Sq. Yard)
	0-4,999	7	12.98
	25,000 and Over	7	11.85
	0-4,999	8	16.67
	5,000-9,999	8	15.46
	10,000-24,999	8	15.30
	25,000 and Over	8	11.72
	0-4,999	9	18.46
	5,000-9,999	9	13.80
	10,000-24,999	9	14.94
	25,000 and Over	9	14.22
	0-4,999	10	24.56
	10,000-24,999	10	18.35
	25,000 and Over	10	15.60

pavement replacements stored in a computer for their study. Tang et al. (1975) used the cost equations proposed by Meredith (1971) in their study. These equations are expressed as functions of invert depths and pipe diameters. They are reproduced as Eqs. (2.1) to (2.3),

$$C=10.98+0.8H-5.98 \quad (2.1)$$

$$D\leqq 3,\ H\leqq 10$$

$$C=5.94+1.166H+0.504HD-9.64 \quad (2.2)$$

$$D\leqq 3,\ H>10$$

$$C=30.0D+4.9H-105.9 \quad (2.3)$$

$$D>3$$

in which C: pipe installation cost ($/linear foot)

H: invert depth (ft)

D: diameter (ft).

Miller (1973, 1975) used an approximation approach to calculate the total costs of drainage pipe systems in which the installation cost was assumed to be two and one half times the material costs. Grigg and O'Hearn (1976) estimated the total project cost by multiplying pipe cost by 1.37 to account for manholes, laterals, engineering, etc.

Merritt and Bogan (1973) used cost functions expressed in different diagrams. Froise (1975) derived costs as a function of pipe diameter and invert depth.

In view of the differences in the methods of estimating costs mentioned above, it is not surprising that some disagreement exists in cost estimates. However, attempts were made in the present study to derive a set of cost functions which includes the information contained in some of the previous studies. For example, it was felt that cost functions expressed as functions of pipe diameters and invert depths would be better representations of actual cost than cost functions expressed as a function of diameters only. Consequently, the cost equations involving pipe diameters and invert depths were developed and are given below as Eqs. (2.4) to (2.6).

$$C=1.93D+1.688H-12.6 \tag{2.4}$$

$$H\leqq 20,\ D\leqq 36$$

$$C=0.696D+2.14H+0.559DH-13.56 \tag{2.5}$$

$$H>20,\ D\leqq 36$$

$$C=3.638D+5.17H-111.72 \tag{2.6}$$

$$D>36$$

In Eqs. (2.4) to (2.6) C: installation costs of the pipe ($/linear foot)

D: diameter (in)

H: invert depth (ft).

The Eqs. (2.4) to (2.6) are based on 1980 values. The cost estimates given by Eqs. (2.4) to (2.6) are compared with those given by three other sources in which the drainage system costs are estimated as a function of diameter and depth. These estimates are listed in Table 2.5. Those previous costs which are not based on 1980 values are updated by Engineering News-Record cost indexes (1980) so that they are comparable. Although differences exist between the cost estimates given by Eqs. (2.4) to (2.6) and previous studies, they are not large and are acceptable.

In the present study, manhole costs are estimated as a function of manhole depths. Meredith (1971) proposed an equation for manhole costs as a function of the depth squared (Eq. (2.7))

$$C_m=250+h^2 \tag{2.7}$$

Merritt and Bogan (1973) used a linear equation to estimate the manhole cost (Eq. (2.8))

Table 2.5 Pipe Cost Comparison

D (in)	H (ft)	Han ($/ft)	Meredith ($/ft)	Δ[1] (%)	Merritt, Bogan ($/ft)	Δ (%)	Froise ($/ft)	Δ (%)	Average ($/ft)	Δ (%)
12	6	20.7	19.5(9.8)	6.2	16.5(10)	25	25.4(18)	-20	20.5	1.1
12	10	27.4	25.8(13)	6.2	23.1(14)	18	35.3(25)	-22	28.1	-2.3
12	15	35.9	33.8(17)	6.2	37.9(23)	-5	46.5(33)	-23	39.4	-8.8
12	20	44.3	41.7(21)	6.2	56.1(34)	-21	59.2(42)	-25	52.3	-15
18	6	32.3	30.4(15.3)	6.2	24.8(15)	-23	29.6(21)	9.1	28.3	14
18	10	39.0	36.8(18.5)	6.0	29.7(18)	-23	39.5(28)	-1.3	35.3	10
18	15	47.5	44.7(22.5)	6.3	42.9(26)	-9.7	50.8(36)	-6.5	46.1	3
18	20	55.9	52.7(26.5)	6.1	70.9(43)	27	64.9(46)	-14	62.8	-11
36	6	67	63.2(31.8)	6.0	57.8(35)	16	55(39)	22	58.7	14
36	10	73.8	69.6(35.0)	6.0	66(40)	12	66.3(47)	11	67.3	9.6
36	15	82.2	77.5(39.0)	6.1	79.2(48)	3.8	83.2(59)	-1.2	80.0	2.7
36	20	90.6	85.4(43.0)	6.1	110.6(67)	-18	101.5(72)	-11	99.2	-8.7
12	25	78.5	75.5(38)	3.9	102.3(62)	-23	76.1(54)	3.2	84.6	-7.2
12	30	95.2	92.2(46.4)	3.3	132(80)	-29	101.5(72)	-6.2	108.5	-12.3
18	25	91.5	94(47.3)	-2.6	108(66)	-15	81(58)	13	94.3	-2.9
18	30	110.0	113(57)	-2.6	143(87)	-23	107(76)	2.8	121	-9.1
36	25	130.6	149(75.1)	-12.3	140(85)	-6.7	124(88)	5.3	137	-4.7
36	30	154.4	175(88.5)	-11.7	176(107)	-12	156(111)	-1.0	169	-8.6
42	10	92.8	95.6(48.1)	-2.9	85.5(52)	8.5	80.4(57)	15	87.3	6.3
60	15	184.1	233.7(117.6)	-21	165(100)	12	138(98)	33	178.9	2.8
72	20	253.6	342(172.1)	-26	239(145)	6.1	215(153)	18	265.3	-4.4

[1]: Δ=(Han-Reference)/(Reference)

$$C_m=140+31h \tag{2.8}$$

Very little information is available about costs of manholes. The data used in the present study are based on 1980 dollar values and have the form given in Eq. 2.9.

$$C_m=259.4+56.4h \tag{2.9}$$

The manhole cost estimates from this equation are compared with other estimates and these are listed in Table 2.6. The costs given by Eq. (2.9) are comparable to those given by Eqs. (2.7) and (2.8). The total cost of the drainage network is the sum of the pipe material cost, laying cost, and the manhole cost. The total drainage system costs used throughout this study are expressed in three Equations (eqs. (2.10) to (2.12)), which are given below, according to the invert depths, pipe diameters, lengths, and manhole costs.

$$C=L\times C_p+L\times(1.93D+1.688H-12.6)+C_m \tag{2.10}$$

$H\leqq 20$, $D\leqq 36$

$$C=L\times C_p+L\times(0.696D+2.14H+0.559DH-13.56)+C_m \tag{2.11}$$

$H>20$, $D\leqq 36$

Table 2.6 Manhole Cost Comparison

H (ft)	Han ($)	Meredith ($)	Δ[1] (%)	Merritt, Bogan ($)	Δ (%)	Ave. ($)	Δ (%)
6	598	568(286)	5	538(326)	11	553	8
10	823	695(350)	18	743(450)	10	719	14
15	1105	944(475)	17	998(605)	11	971	14
20	1387	1291(650)	7	1254(760)	11	1273	9

[1]:Δ=(Han−Reference)/(Reference)

$$C=L\times C_p+L\times 3.638D+5.17H-111.72)+C_m \quad (2.12)$$

$d>36$

in which L : length of the pipe (ft)

C_p: unit cost of pipe material ($/linear foot)

D : diameter of the pipe (in)

C_m: manhole cost ($)

Detention storage basin costs were calculated by using the unit cost of excavation and concrete surface, fence, and the cost of land. The prices used are applicable to Lafayette area and are typical. They may have to be altered according to local conditions if the cost study is to be repeated in other areas.

2.5 Parameter Values Used in the Study

Some of the parameters of ILLUDAS were held constant when they were applied to different watersheds. These parameters were held constant either because variation in them did not drastically affect the output, or they could be measured accurately, or they could be easily obtained from readily available references. These parameters which were held constant are listed in Table 2.7. The hyetographs used in the cost analysis in Chapter V are taken from Rao and Chenchayya (1974) and are given in Table 2.8. Also included in Table 2.8 is the built-in Huff distribution (1967) which is the default option in ILLUDAS.

Table 2.7 Constant Input Used in the Study

	ILLUDAS	
Input Parameter	Upper Ross-Ade Watershed	Bar Barry Heights
Area (acres)	29.1	121.9
Soil Group	B	B
Manning's n	0.013	0.013
Time Increment (min)	5	5
Routing Scheme	Time Lag	Time Lag

Table, 2.8 Percentage Cumulative of 10 Minutes Rainfall Data in West Lafayette, Indiana (From Rao and Chenchayya (1974))

%	I Quartile			II Quartile			III Quartile			IV Quartile			Huff
Time	10%	50%	90%	10%	50%	90%	10%	50%	90%	10%	50%	90%	Distribution
0	0	0	0	0	0	0	0	0	0	0	0	0	0.0
4	37	21	10	10	3	3	17	8	5	6	4	3	9.6
8	58	40	22	18	11	8	26	12	9	14	8	7	21.0
12	72	48	35	25	14	12	30	17	12	20	12	9	32.7
16	80	60	45	33	21	18	33	19	14	28	16	12	43.0
20	87	68	54	42	29	23	35	22	17	35	20	15	51.2
24	90	76	60	57	39	29	37	25	19	40	25	18	58.3
28	92	79	67	70	48	40	39	27	22	46	30	22	63.1
32	95	81	70	82	52	48	41	30	25	50	35	25	67.2
36	96	85	75	90	62	58	44	35	28	54	38	28	70.6
40	97	87	78	93	73	62	46	38	30	58	40	30	73.5
50	98	90	84	96	88	80	61	46	40	64	48	39	79.5
60	98.5	94	89	98	93	87	92	73	58	68	55	46	84.2
70	99	95	93	98.5	95	92	96	85	77	73	63	58	88.5
80	99.5	97	95	99	98	95	97	93	86	88	74	72	92.5
90	99.8	98	97	99.5	99	98	99	97	95	98	90	92	96.3
100	100	100	100	100	100	100	100	100	100	100	100	100	100.0

CHAPTER III

ILLUDAS

The ILLUDAS model was originally developed for simulation of urban runoff from single storm events. The runoff from both grassed and paved areas are estimated in this model.

The paved area hydrograph is calculated in ILLUDAS by the linear time-area method. The travel time is found by Manning's equation and the initial abstraction is subtracted from the initial hyetograph increment to obtain the paved area supply rate (PASR). The final hydrograph is determined by assuming that the first increment of runoff results from the first increment of area and first PASR. The second runoff area increment results from the first PASR on the second area and the second PASR on the first area increment and so on. These calculations may be expressed in the matrix notation as follows.

$$[Q]=[PASR][PA] \tag{3.1}$$

in which

$$[Q]=\begin{bmatrix} Q_1 \\ Q_2 \\ \cdot \\ \cdot \\ Q_n \end{bmatrix} \qquad [PA]=\begin{bmatrix} PA_1 \\ PA_2 \\ \cdot \\ \cdot \\ PA_n \end{bmatrix}$$

$$[PASR]=\begin{bmatrix} PASR_1 & 0 & 0 & 0 \\ PASR_2 & PASR_1 & 0 & 0 \\ \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot \\ PASR_n & \cdot & \cdot & PASR_1 \end{bmatrix}$$

The grassed area hydrograph which consists of runoff from the grassed area and the supplemental paved area is considered by ILLUDAS but it was ignored in the RRL method (Watkins (1962)) from which ILLUDAS evoloved. The procedure is quite similar to that of the paved area hydrograph. The travel time is found by Izzard's equation (Izzard (1946)) and linearity is assumed for the time-area curve. The grassed area supply rate (GASR) is determined by adding the hyetograph to the supplemental paved area runoff (SPARO). These areas typically include roofs, sidewalks, etc. The infiltration in the watershed is calculated by using Horton's equation (Horton (1940), Chow (1964)). The grassed area hydrographs are then derived by:

$$[Q]=[GASR][GA] \tag{3.2}$$

in which

$$[Q]=\begin{bmatrix} Q_1 \\ Q_2 \\ \cdot \\ \cdot \\ Q_n \end{bmatrix} \qquad [GA]=\begin{bmatrix} GA_1 \\ GA_2 \\ \cdot \\ \cdot \\ GA_n \end{bmatrix}$$

$$[GASR]=\begin{bmatrix} GASR_1 & 0 & 0 & 0 \\ GASR_2 & 0 & 0 & 0 \\ \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot \\ GASR_n & \cdot & \cdot & GASR_1 \end{bmatrix}$$

Once the grassed and paved area hydrographs are determined, they are added together and to the upstream hydrographs. The sewer is sized at this stage by using Manning's equation and the peak flows. The smallest allowable commercially available pipe which can carry the flow equal to or larger than the peak flow is selected. The hydrograph is routed further downstream to another design point by one of two routing methods. They are, (1) the time shift or time lag method, and (2) storage routing in which the implicit solution of the continuity equation is used. The process is repeated until the final outlet is reached.

In the design mode, ILLUDAS is used to calculate the required channel or sewer diameters. In using ILLUDAS it is necessary to subdivide the watershed into subbasins served by different sewer branches and reaches. The data needed to use ILLUDAS include the rainfall magnitude and temporal distribution, the antecent moisture condition, the hydrologic soil group, and the percentage of paved and grassed areas. In the evaluation mode, the pipe, culvert or open ditch dimensions, their lengths, slopes, values of Manning's n, etc. are also needed. Maps, aerial photos and drainage system drawings of the basin may be needed to extract the branch, reach, and subbasin information. Even so, the data requirements of ILLUDAS are moderate compared to those of other urban runoff models.

The user's manual and the computer deck of ILLUDAS may be obtained from the Illinois State Water Survey (Terstriep and Stall (1974)). The program has been tested and the results have been found to be satisfactory. Burke (1979) compared ILLUDAS with two other methods on two watersheds and concluded that ILLUDAS is accurate, flexible, easy to apply, and moderate in data requirements.

ILLUDAS is being continuously improved. In December, 1978, the flow routing algorithm was expanded by the

Illinois State Water Survey. The latest version of ILLUDAS dated October, 1979, has two routing options, a time shift of the entire hydrograph, and storage routing using the implicit solution method. The water quality algorithm of SWMM is also being adapted in a version of ILLUDAS known as QUAL-ILLUDAS (Terstriep et al. (1978)). Han and Delleur (1979) modified ILLUDAS for continuous simulation and added a single storm quality simulation routine to the ILLUDAS model.

CHAPTER IV

DYNAMIC PROGRAMMING ALGORITHM

The dynamic programming algorithm used in the least cost design model may be best explained by a simple example. However, before the discussion of the example, the notation used is described.

N: total number of manholes in the sewer system

n: index of manholes (stages) or "stages" of dynamic programming which correspond to various subproblems contained in the overall problem.

ξ_n: set of manholes directly above manhole n; i.e, the set of manholes which have pipes leading directly into manhole n. For the uppermost manhole or the manhole without any upstream manhole directly connected, ξ_n=[ ].

E_n: elevation of outflow pipe from manhole n, there will be several possible elevation values (states).

E_n°: pipe elevation which exits from manhole n at the point where it enters the next manhole.

$Cost_i(E_i, E_i°)$: cost of pipe (material and installation costs) from manhole i to the next downstream manhole when the elevation of the pipe exiting manhole i is E_i and elevation of pipe entering the downstream manhole is $E_i°$ plus cost of installing manhole i; i is a dummy index of manhole.

S: set of scanned or examined manholes. Initially this set contains all manholes which have no upstream manholes, that is all manholes for which $.x_n$=[ ].

U: set of unscanned manholes, Initially this set contains all manholes in which at least one pipe enters from another manhole. In the present program, there is a maximum of eight pipes entering into a manhole.

$f_n(E_n)$: minimum cost of constructing the entire sewer system above manhole n when the outflow elevation from manhole n equals E_n. Initially this is set to zero for the upstream end of a system, i.e, no pipes enter manhole n, only a pipe leaves.

The recursive equation for the dynamic programming algorithm is based on minimizing the costs subject to the constraints as given in Eq. (4.1)

$$f_n(E_n) = \min_{\substack{E_i, E_i^{\circ}\ i \in \xi_n \\ E_i^{\circ} \leq E_i\ i \in \xi_n \\ \text{acceptable velocities}}} \left[\sum_{i \in \xi_n} Cost_i(E_i, E_i^{\circ}) + f_i(E_i) \right] \quad (4.1)$$

The dynamic programming technique is based on the following principle of optimality (Bellman (1957)).

"An optimal policy has the property that whatever the initial decisions are, the remaining decisions must constitute an optimal policy with regard to the state resulting from the first decision".

Although the principle of optimality is easy to understand, it has not been widely applied and used in sewer system design because of a lack of understanding of the dynamic programming algorithm. Houck (1979) has developed a simple, straight-forward computer program which comprises all the steps necessary to solve an engineering problem by dynamic programming. This is the program used in the present study.

In the study herein, the solution steps are as follows:

(1). Choose a manhole j from U such that ξ_j is a subset of S. This is to assure that $f_i(E_i) \forall\ i \in \xi_j$ are already known.

(2). For each potential elevation from manhole j, E_j, examine all possible and feasible combinations of upstream pipe elevations and entering pipe elevations to determine the least cost design. This is the value of $f_n(E_n)$.

(3). Once all the potential elevations are examined and the costs are calculated, move manhole j from the set U to the set S. If set U is empty go to the next step; otherwise, go to step (1) and continue.

(4). Print out the optimal solutions and least cost alternatives.

A very simple hypothetical sewer system is used below to demonstrate the use of the dynamic programming algorithm. The simplicity of the system allows hand calculations and hence the calculations can be explained step-by-step. The example system includes only two pipe segments connecting three manholes in a straight line as shown in Fig. 4.1. Manhole 1 is at the upstream end of the system and manhole 3 is the final outlet. Manhole 1 is 100 ft from manhole 2 and manhole 2 is 200 ft from manhole 3. The ground surface is considered to be flat and has an elevation of 20 ft above datum. All pipes are required be at least 5 ft below the ground level. The required minimum and maximum velocities of flow in these pipes are 2 fps and 8 fps, respectively. The commercially available pipe sizes are 6, 9, 12, 15, 18, 21, and 24 inches. The cost functions discussed in Chapter

II are used in this example. The design inflows have been determined to be 5 ft^3/sec for manhole 1 and 6 ft^3/sec for manhole 2. In the actual least cost design method discussed below, these quantities are estimated by using the hydrologic model, ILLUDAS. For simplicity, only several discrete pipe elevations at each manhole E_n have been used in computations. Many more possible elevations could be examined with the computer.

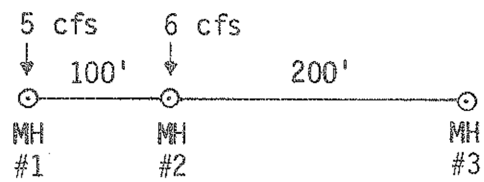

Figure 4.1 Layout of the Example System

According to the notations defined earlier, the values given below are used in this example.

N = total number of manholes in the system, 3.

ξ_n = the set of manholes which have pipes leading directly into manhole n; ξ_1=[ ], ξ_2=[1], and ξ_3=[2].

S = the set containing all manholes which have no upstream manholes, S=[1].

U = the set containing all manholes in which at least one pipe enters from another manhole, U=[2,3].

E_n = elevation of outflow pipe from manhole n. In the example, these elevations are constrained by maximum and minimum depths required to lay the pipes. E_1=[13,14,15], i.e., there are only three possible elevations of 13, 14, and 15 ft to lay the pipe at manhole 1. Similarly, E_2=[11,12,13,14], and E_3=[8,9,10,11,12,13].

First the manhole 2 is considered so that ξ_2=[1]. For each potential elevation from manhole 2, i.e., E_2=[11,12,13,14], the possible feasible combinations of sewer elevations from manhole 1 to manhole 2 are (15,14), (15,13), (15,12), (15,11); (14,14), (14,13), (14,12), (14,11); (13,13), (13,12), (13,11). Costs are estimated for these combinations by using cost functions to determine the values of $Cost_1(E_1, E_1°)$. The combinations are checked to see if they violate any constraints. If a combination violates any of the constraints, it will not provide a feasible solution. The results of calculation for this stage are tabulated in Table 4.1. Several combinations which are either infeasible or which violate the constraints are specially marked in the table. The minimum cost for the state variable and the optimal elevations for that stage are listed in the last two columns of the table.

After the optimal solution of stage 1 is obtained, manhole 2 is moved from U to S and now manhole 3 can be considered. The state variable set for manhole 3 is

Table 4.1 Calculation of Stage 1

$Cost_1(E_1, E_1^o)$

State $[E_2]$	(15,14)	(15,13)	(15,12)	(15,11)	(14,14)	(14,13)	(14,12)	(14,11)	(13,13)	(13,12)	(13,11)	$f_2(E_2)$	$(E_1, E_1^o)^*$
14	15" 3861	X	X	X	XXX	X	X	X	XXX	X	X	3861	(15,14)
13	XX	12" 3248	X	X	XX	15" 4086	X	X	XXX	X	X	3248	(15,13)
12	XX	XX	12" 3389	X	XX	XX	12" 3473	X	XXX	15" 4312	X	3473	(14,12)
11	XX	XX	XX	XXXX	XX	XX	XX	12" 3614	XXX	XX	12" 3698	3614	(14,11)

X: entering elevation is below state variable being considered

XX: entering elevation is above state variable being considered

XXX: flow velocity is below the minimum requirement

XXXX: flow velocity is above the maximum limitation

E_3=[8,9,10,11,12,13]. The feasible combinations of sewer line elevations are (14,13), (14,12), (14,11), (14,10), (14,9), (14,8); (13,13), (13,12), (13,11), (13,10), (13,9), (13,8); (12,12), (12,11), (12,10), (12,9), (12,8); (11,11), (11,10), (11,9), (11,8). The results of computations for this state are summarized in Table 4.2. The cost of each feasible combination which satisfies the constraints is shown in Fig. 4.2. The least cost design is determined by following a backward search from manhole 3 to manhole 2 and from manhole 2 to manhole 1. The final optimal design has elevations of 15, 13, and 10 ft for manholes 1,2, and 3 respectively with pipe diameters of 12" and 18". The total least cost is approximately $ 13,100.

The simple example discussed above demonstrates the computational steps in the computer program used in this study. As one would expect, hand calculations become impractical with the increasing number of state variables. However, the computation is possible with digital computers. It is important to note that the design inflows play a significant role in these computations. Hence it is necessary to evaluate them accurately. A good hydrologic model is thus essential for successful use of this method.

Table 4.2 Calculation of Stage 2

$Cost_2(E_2,E_2^o)$

State $[E_3]$	(14,13)	(14,12)	(14,11)	(14,10)	(14,9)	(14,8)	(13,13)	(13,12)	(13,11)	(13,10)	(13,9)	(13,8)	(12,12)	(12,11)	(12,10)	(12,9)	(12,8)	(11,11)	(11,10)	(11,9)	(11,8)	$f_3(E_3)$	$(E_2,E_2^o)^*$
13	21" 10704	X	X	X	X	X	XXX	X	X	X	X	X	X	X	X	X	X	X	X	X	X	10704	(14,13)
12	XX	21" 10930	X	X	X	X	XX	21" 11087	X	X	X	X	XXX	X	X	X	X	X	X	X	X	10930	(14,12)
11	XX	XX	18" 9467	X	X	X	XX	XX	21" 11324	X	X	X	XX	21" 11493	X	X	X	XXX	X	X	X	9467	(14,11)
10	XX	XX	XX	18" 9692	X	X	XX	XX	XX	18" 9861	X	X	XX	XX	21" 11718	X	X	XX	21" 11887	X	X	9692	(14,10)
9	XX	XX	XX	XX	XXXX	X	XX	XX	XX	XX	18" 10686	X	XX	XX	XX	18" 10255	X	XX	XX	21" 12112	X	10086	(13,9)
8	XX	XX	XX	XX	XX	XXXX	XX	XX	XX	XX	XX	XXXX	XX	XX	XX	XX	18" 10480	XX	XX	XX	18" 10649	10480	(12,8)

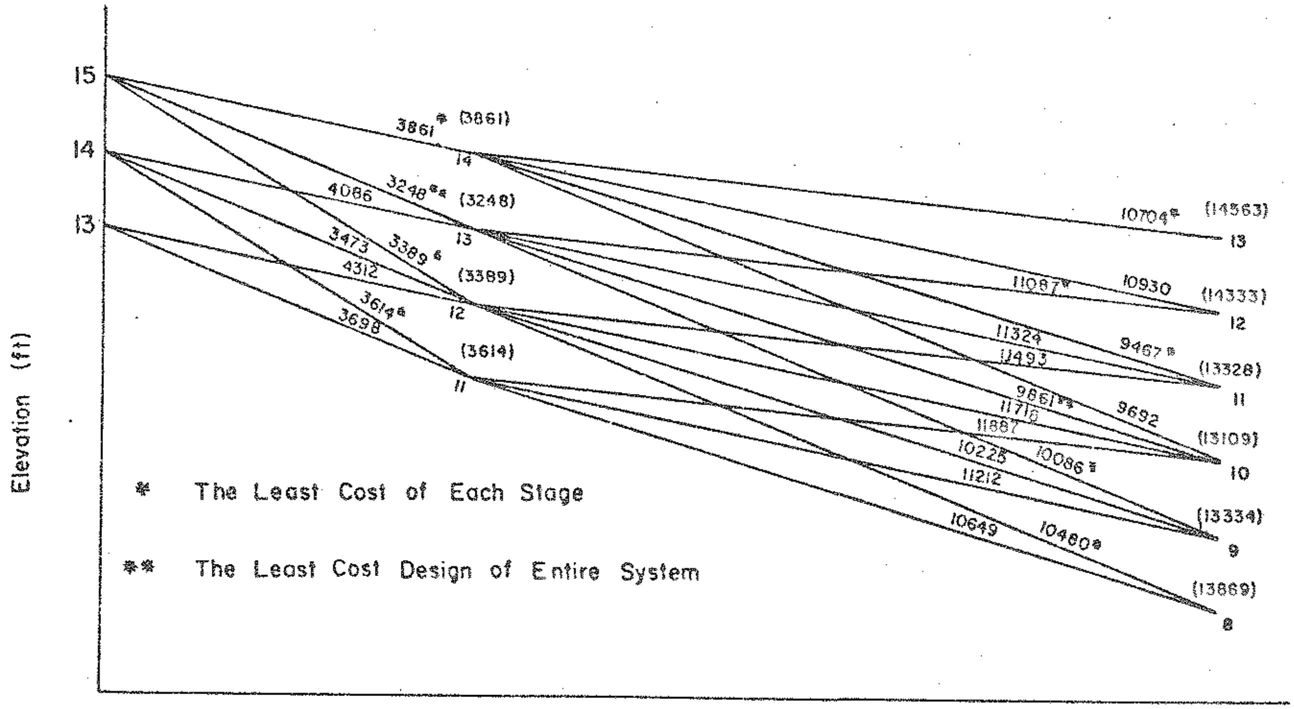

Figure 4.2 Costs for Various Combinations

CHAPTER V

RESULTS FROM THE LEAST COST DRAINAGE SYSTEM DESIGN MODEL

5.1 Introduction

Data from the Upper Ross-Ade Watershed and the Bar Barry Heights Subdivision were chosen to test the model. The layout of the watersheds and other related information have been described in Chapter II. The storm characteristics for the area investigated, including dimensionless hyetographs, are given by Rao and Chenchayya (1974). The depth-frequency-duration information for the area are summarized by Burke (1979). The cost data used in the study are discussed in Chapter II.

The effects of variation of the more important design parameters on the system costs were investigated. The effects of variation of input parameters on the design of the drainage systems have been investigated by Burke et al. (1980). They examined the variation of parameters used in the ILLUDAS model which includes the duration of rainfall, return period of rainfall, and antecedent moisture condition on runoff. They also investigated the variations in the model output due to variations in the temporal distribution of the rainfall. These investigations are extended in this study in which the effects of variations of input parameters on the system costs are considered.

5.2 Results

5.2.1 Upper Ross-Ade Watershed

Two design options were considered for the Upper Ross-Ade Watershed. These are, a sewer network without detention storage and a network with detention storage in the reach 1-3 (see Fig. 2.4). The relationship between system costs and input parameters, and other aspects were investigated for both of these options and the results are presented below.

(a) Costs of Drainage Systems With and Without Detention Storage and Their Variation With Design Storm Durations and Frequencies

The least cost design model was used to determine the variation in cost with design parameters. Five design storm return periods (2, 5, 10, 25, and 100 years), and six design storm durations (5, 10, 20, 30, 60, and 120 minutes) were used in this study. Two drainage systems, one of which is the pipe network and another which includes a detention storage basin also, were used. The antecedent moisture condition class is fixed as class 2, which occurs most frequently in the Lafayette area (Gray and Cogo (1980)). The detention storage volume was arbitrarily selected to be 10,000 cubic feet. The costs of the systems for these conditions are tabulated in Table 5.1. In most of these instances (Table 5.1), the cost of the system with detention storage is less than that of the system without it. The advantages of including detention storage are obvious from the results presented in Table 5.1. Those few instances where the system without detention storage costs less than the system with detention storage are due to the arbitrary selection of the 10,000 ft^3 detention storage volume. In a later section, the optimal selection of detention storage volume will be discussed.

The variation of drainage system costs with storm durations is shown in Fig. 5.1. The storm duration for which the drainage system cost is maximum may be called the "critical duration." A duration between 10 to 20 minutes is acceptable as the critical duration for the Upper Ross-Ade Watershed. Although the results presented in Fig. 5.1 are strictly valid only for the Ross-Ade Watershed, they provide a method of determining "critical" storm durations for the drainage systems. It is important to note that less expensive systems may be designed by altering the critical duration but by keeping the same storm frequencies.

The relationship between drainage system costs and return periods of design storms of 20 minute duration (which is close to the critical duration) are presented in Fig. 5.2. Costs vary rather rapidly with the small return periods (2, 5, and 10 years), but not very much with return periods

Table 5.1 Variation of Costs With Storm Duration for Several Return Periods, Upper Ross-Ade Watershed

	2 yrs		5 yrs		10 yrs		25 yrs		100 yrs	
	Design I	Design II	Design I	Design II	Design I	Design II	Design I	Design II	Design I	Design II
5 min	104,105	128,868	115,534	156,776	124,625	171,178	145,869	190,121	171,472	194,725
10 min	113,792	153,452	142,242	184,998	160,822	194,659	174,702	194,725	207,159	231,337
20 min	130,314	169,566	155,624	188,518	173,055	194,659	206,890	210,245	208,264	234,064
30 min	115,571	144,506	147,014	174,570	174,258	194,594	207,449	194,659	205,602	231,337
60 min	110,390	142,556	129,829	158,206	155,671	183,976	175,906	194,594	203,027	196,894
120 min	99,943	126,026	111,292	128,933	114,185	138,948	130,885	153,452	167,966	182,167

I: design with a detention storage basin of 10,000 cubic feet
II: design without detention storage basin

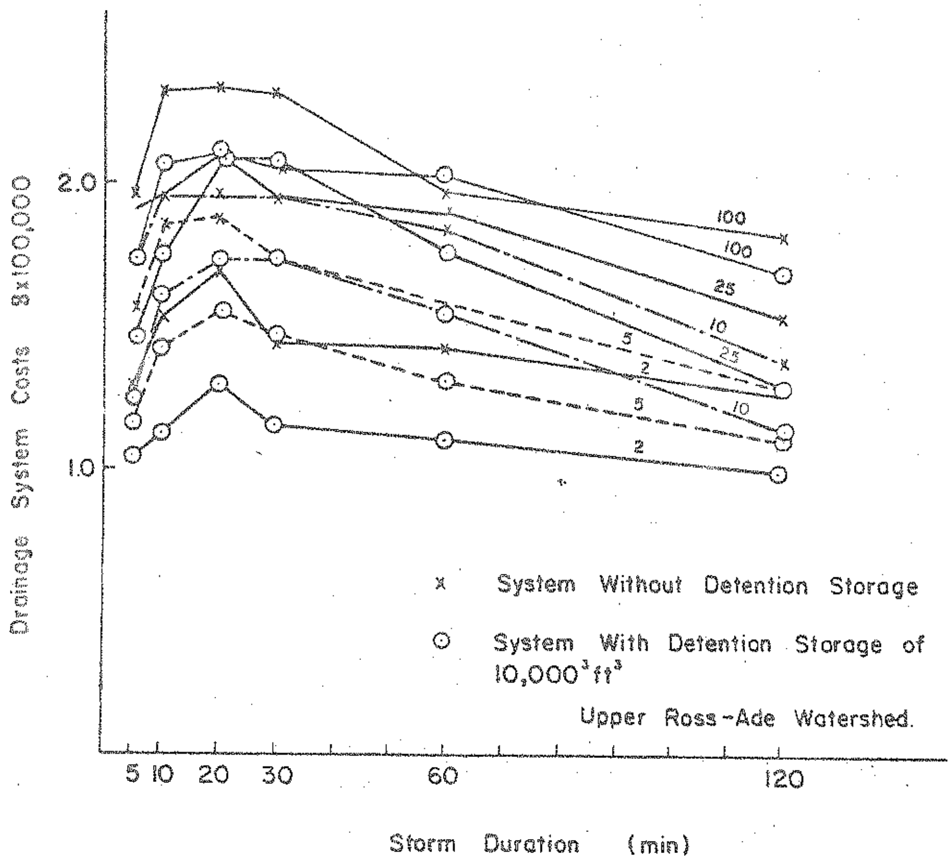

Figure 5.1 Relationships Among Drainage System Costs and Design Storm Durations and Frequencies, Upper Ross-Ade Watershed

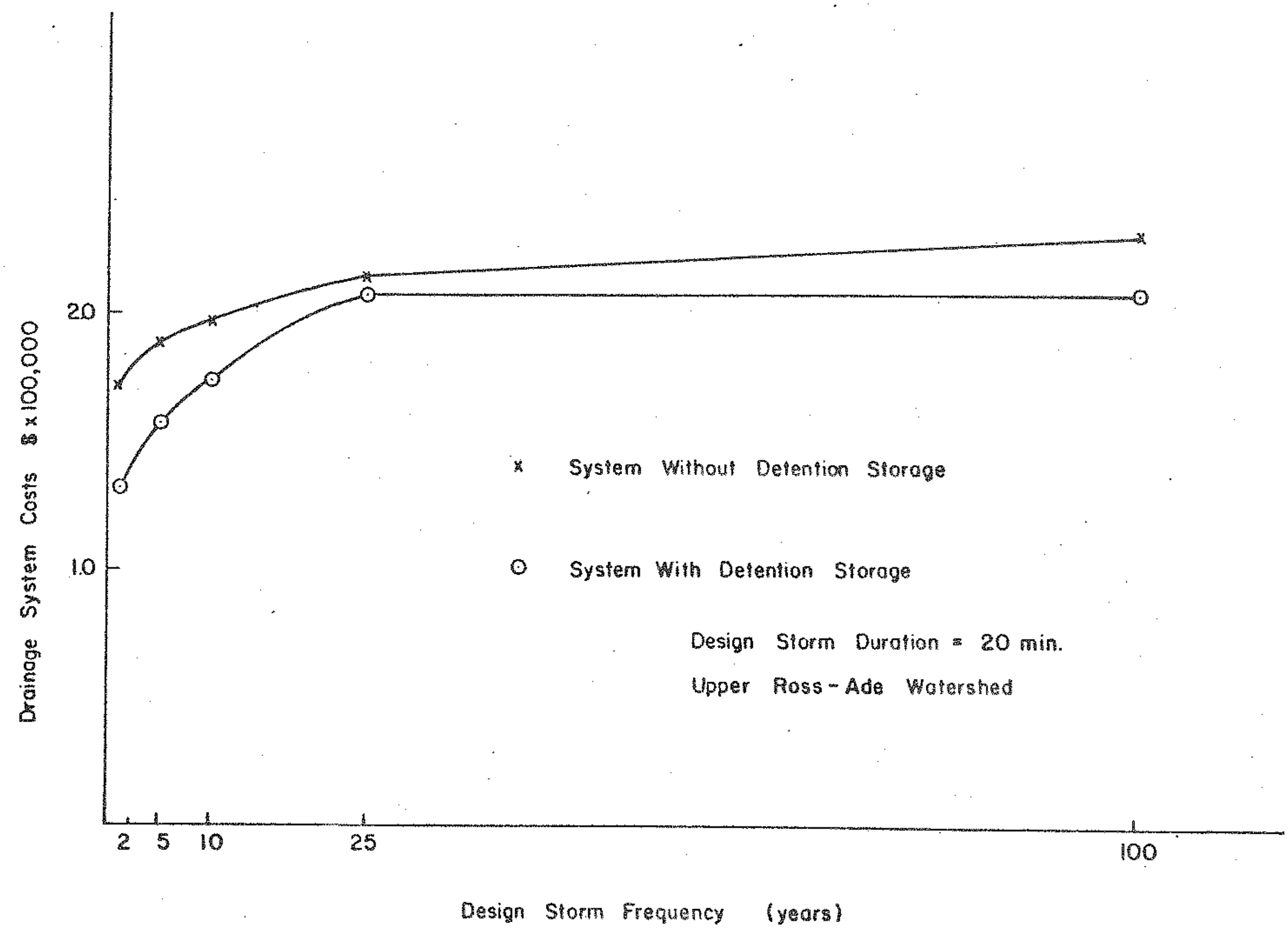

Figure 5.2 Relationships Between Drainage System Costs and Design Storm Frequencies, Upper Ross-Ade Watershed

higher than 25 years. These findings agree with those of other investigators (Rawls and MCuen (1978), Grigg and O'Hearn (1976)). The percent differences in costs of drainage systems designed by using a 10-year design storm or a 2 or a 5-year design storm can be quite large. Examples of these differences are shown in Table 5.2 for the Upper Ross-Ade Watershed. These increased costs range from 3% to 20% when a 10-year storm instead of a 5-year storm is used in a design with detention storage, and from 4% to 16% in a design without detention storage. The costs increase from 14% to 51% for a design with detention storage, and from 11% to 35% for a design without detention storage if a 10-year design storm is used instead of a 2-year design storm. These aspects should be considered when design storm return periods are selected.

(b) Variation of Drainage System Costs with AMC

The antecedent moisture condition is an important design parameter in ILLUDAS. It is clear that AMC 4 would provide the most conservative design. The drainage system costs were estimated in the present study for different AMC values and the "critical" duration of 20 minutes, and the most commonly adopted return period of 5 years (Rogers (1980)). The variation in costs with AMC is shown in Fig. 5.3. The drainage system designed by using AMC 4 costs the maximum as expected. It is also interesting to note that for AMC 4, the design option based on detention storage can only save

Table 5.2 Percent Difference In Cost of Drainage Systems Designed by Using Storms of Different Return Periods, Upper Ross-Ade Watershed

	% difference between			
	10 and 5 years		10 and 2 years	
	Design I	Design II	Design I	Design II
5 min	7.9	9.2	19.7	32.8
10 min	13	5.2	41.3	26.9
20 min	11.2	3.3	32.8	14.8
30 min	18	11.5	50.8	34.7
60 min	19.9	16.3	41	29.1
120 min	2.6	7.8	14.2	10.3

I: design with a detention storage basin with 10,000 cubic feet

II: design without detention storage basin

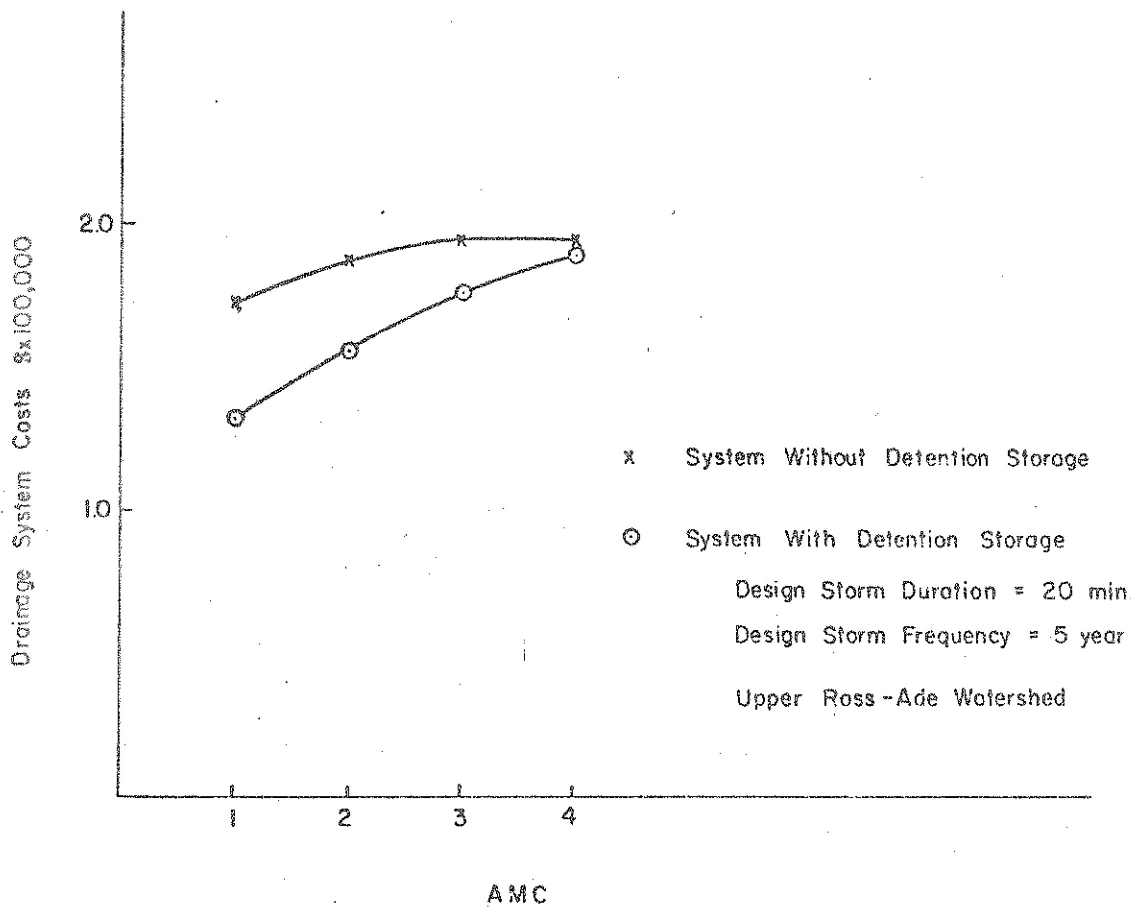

Figure 5.3 Variation in Drainage System Costs With AMC, Upper Ross-Ade Watershed

less than 3% over the design without detention storage as compared to a savings of 30% if AMC 1 is used.

(c) Sensitivity of Drainage System Costs to Input Hyetographs

The sensitivity of costs of drainage systems to input design hyetographs is discussed below. In ILLUDAS the design hyetographs may be read in or they may be computed by using the median percentage dimensionless mass curve for first quartile storms generated by Huff (1967). This mass curve was estimated by using some rainfall data measured in East Central Illinois.

In drainage design, dimensionless hyetographs at the extremes such as those corresponding to 10% and 90% exceedences and to 50% are of interest. Consequently the 10%, 50%, and 90% dimensionless mass curves developed from West Lafayette 10-minute rainfall data for the four quartiles (Rao and Chenchayya (1974)) are used in the study. These dimensionless mass curves are tabulated in Chapter II.

Costs of drainage systems - each of which is a least cost system - were estimated by using these three percentage curves and all four quartile storms for systems both with and without detention storages. These results are shown in Fig. 5.4. These results indicate that the first quartile storms give the costliest designs, and the systems without detention storage cost more than those which include

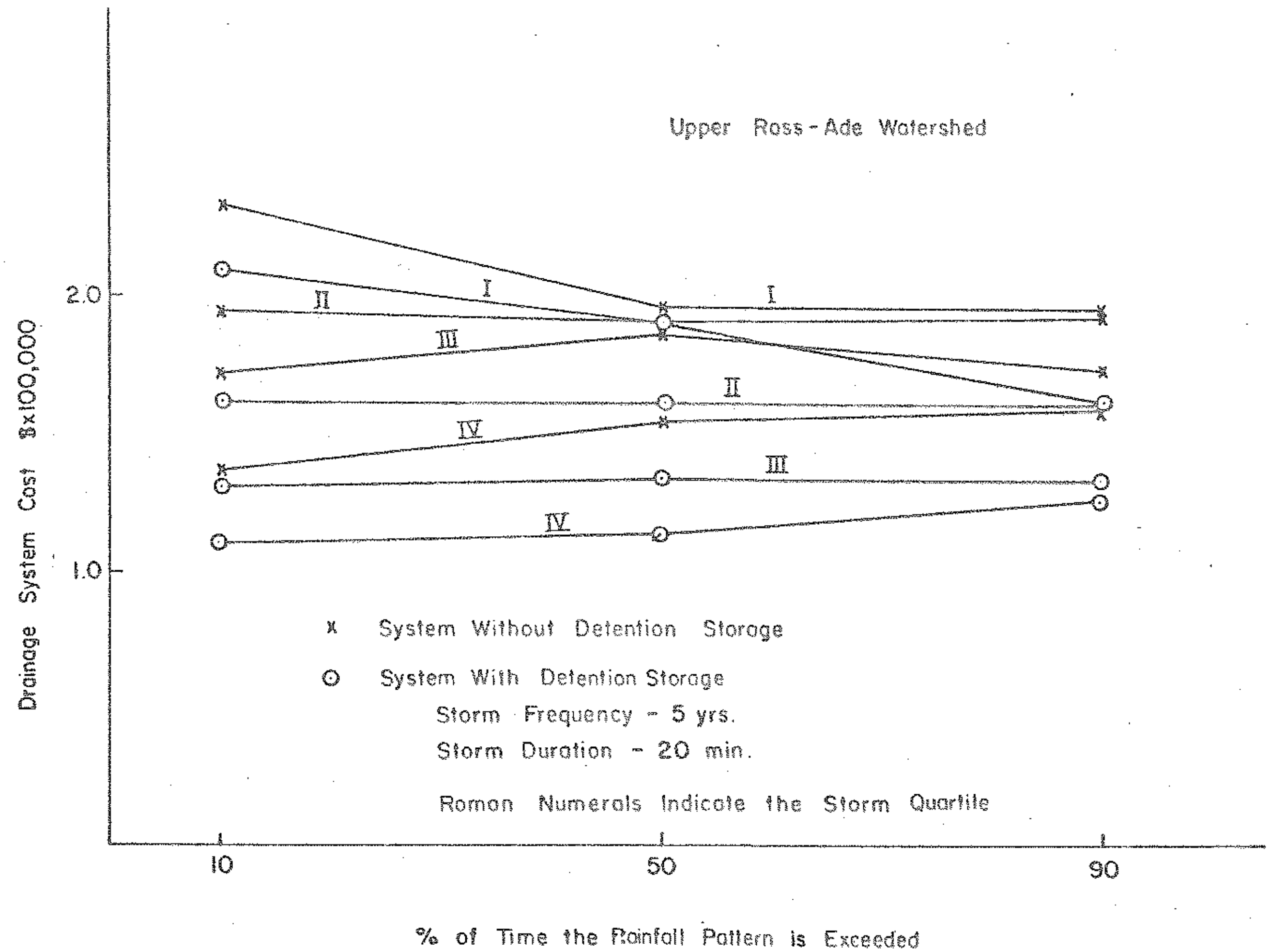

Figure 5.4 Variation of Costs With Storm Exceedence Percentages for Four Quartile Storms

detention storage. The costs reduce with increasing storm-quartile values.

Costs estimated by using the median curve for the four quartile storms and also the Huff first quartile distribution which is the default option in ILLUDAS are plotted in Fig. 5.5 for designs with and without detention storage. The hatched areas in Fig. 5.5 represent the drainage system costs estimated by using the Huff distribution. These results again indicate that the largest costs occur with first quartile storms and steadily decrease with the second, third, and fourth quartile storms.

Another interesting result is that the cost of drainage systems is greater for the first and second quartile storms estimated from West Lafayette data than the system costs estimated by using the Huff first quartile median dimensionless hyetograph. Consequently, whenever possible, the dimensionless hyetographs estimated by using local data should be used.

(d) Optimal Detention Storage Volumes

Several methods for sizing the detention storage basin have been developed (Rao (1975), Wycoff and Singh (1976), Burke and Gray (1980)). Those methods were developed by using hydrologic considerations only. In these methods detention storages are not sized by using cost criteria. The optimal size of a detention storage basin based on the

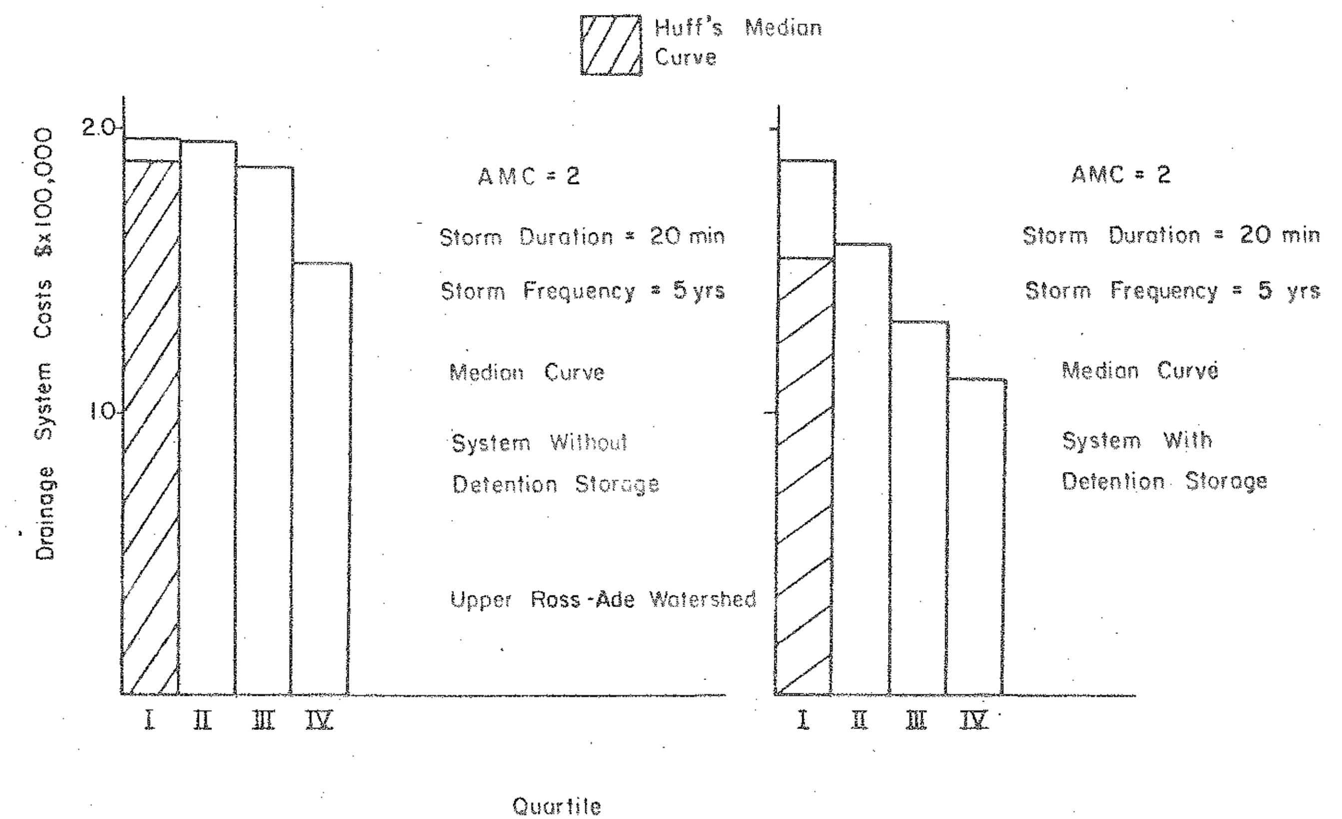

Figure 5.5 Variation of Costs With the Four Quartile Storms and With Huff's Median, First Quartile Storm, Upper Ross-Ade Watershed

costs of drainage systems may be obtained by using the least cost model developed in the present study. The procedure is as follows.

The detention storage volume is specified along with other design parameters such as the AMC. The least cost design along with its estimated cost is obtained by the model. The detention storage volume is changed and the procedure is repeated. The variation of detention storage volume with the associated costs may be plotted as shown in Figs. 5.6 through 5.8 for various combinations of design storm durations (10, 20, and 30 minutes) and frequencies (2, 5, and 10 years). The detention storage volume associated with the least system cost, shown in hollow circles in Figs. 5.6 through 5.8, may be easily selected. The drainage system costs shown in Fig. 5.6 through 5.8 become constant after a certain detention storage is reached. This is due to the fact that the outflow from the detention storage is automatically fixed at a constant minimum value by ILLUDAS.

5.2.2 Bar Barry Heights Subdivision

The relationships between drainage system costs and design parameters were investigated by using data from Bar Barry Heights Subdivision also. Two design options were included in the study: (1) a sewer pipe network without any detention storage and, (2) a sewer pipe network with two detention storage basins located between manholes 10 and

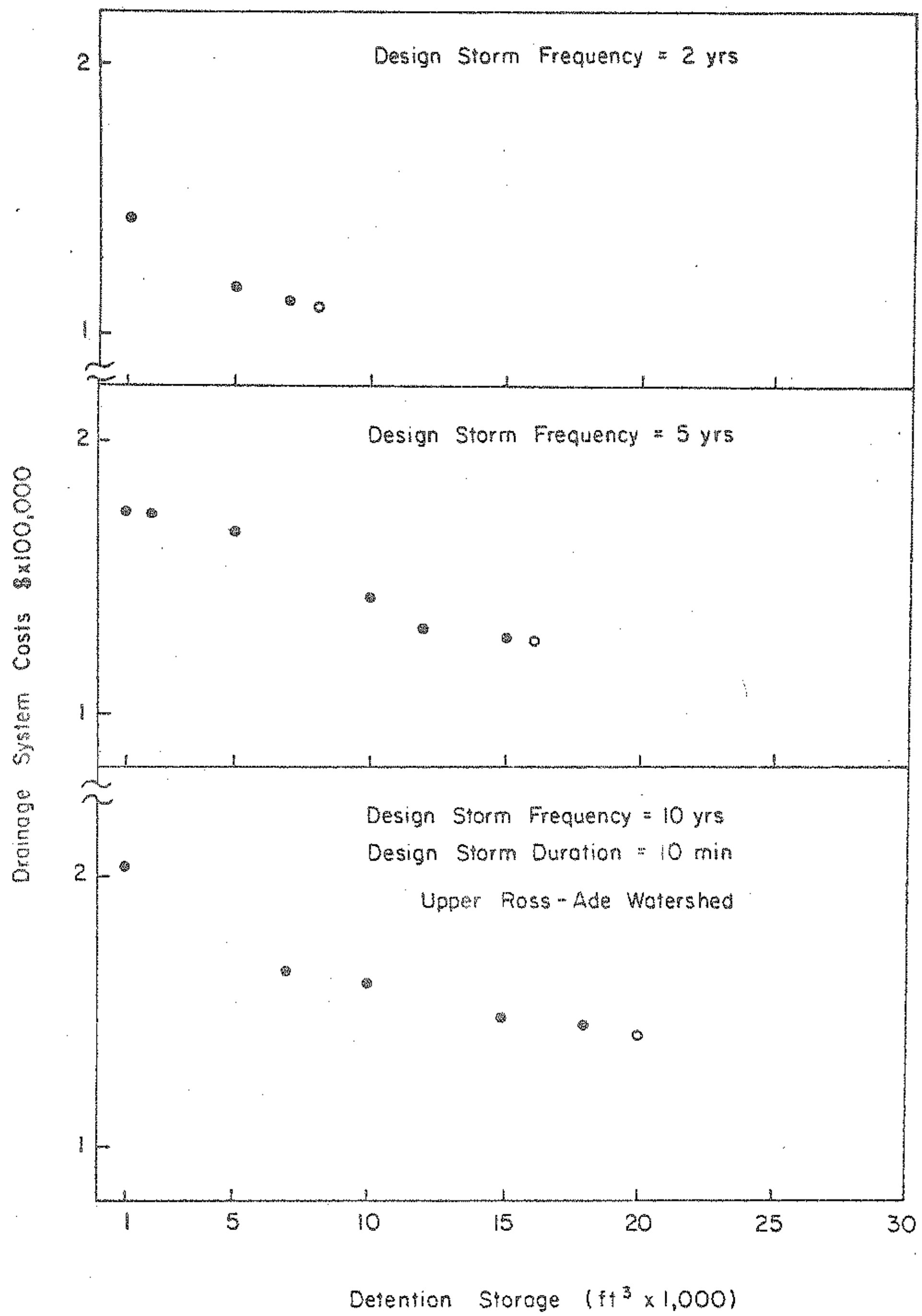

Figure 5.6 Variation of Costs With Detention Storage Volumes, 10 Minutes Storm Duration, Upper Ross-Ade Watershed

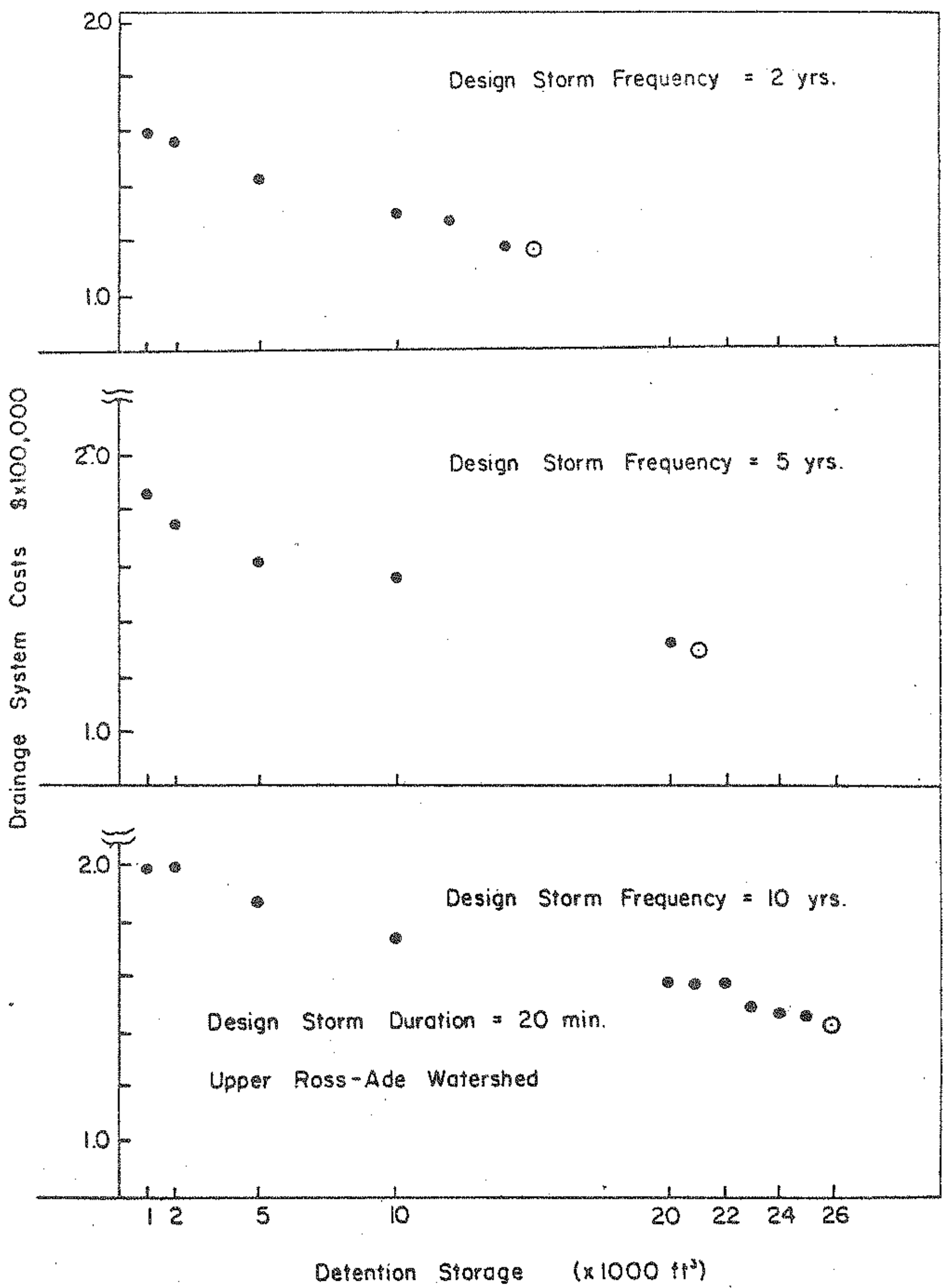

Figure 5.7 Variation of Costs With Detention Storage Volumes, 20 Minutes Storm Duration, Upper Ross-Ade Watershed

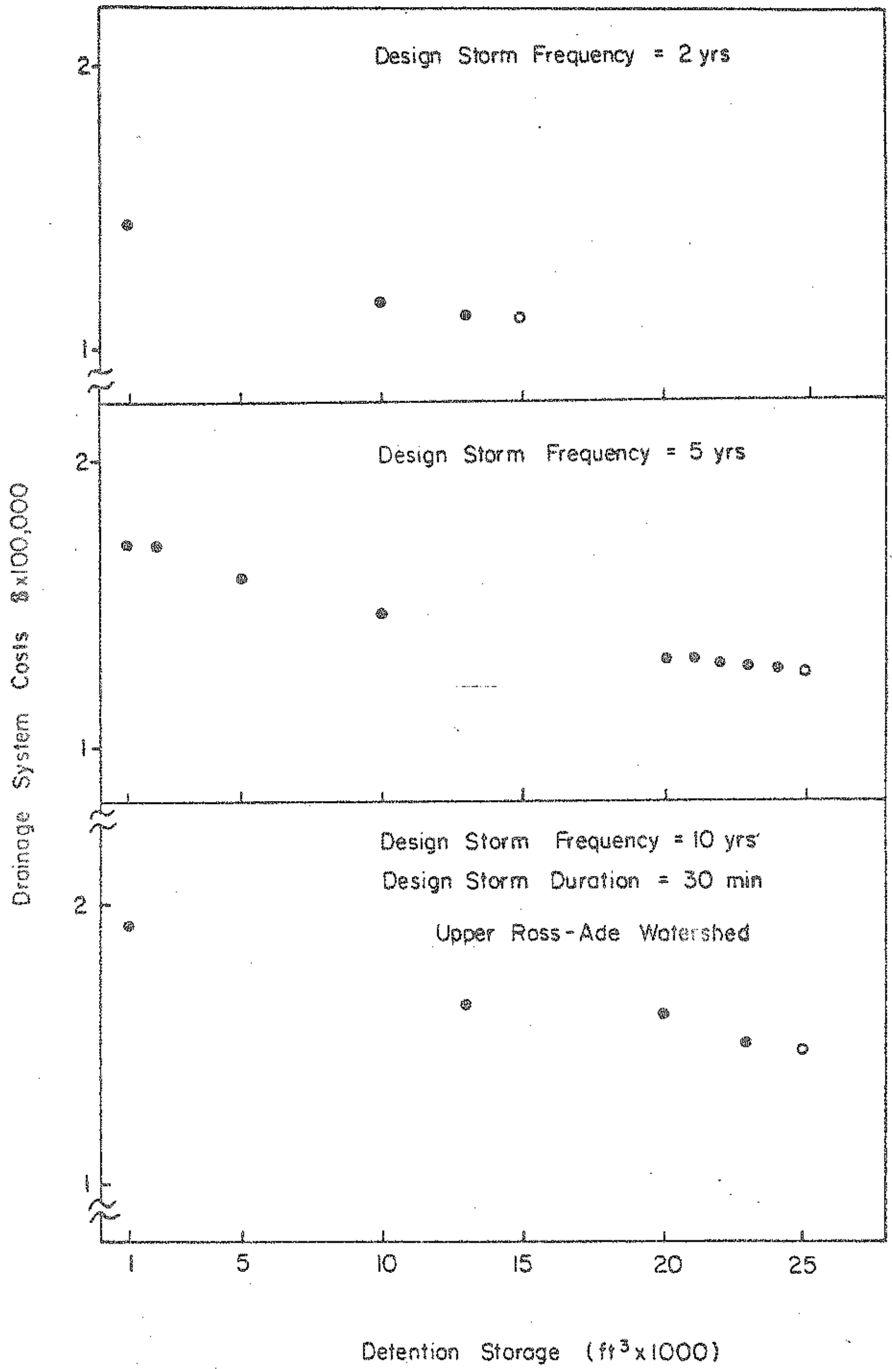

Figure 5.8 Variation of Costs With Detention Storage Volumes, 30 Minutes Storm Duration, Upper Ross-Ade Watershed

10A, and between manholes 58 and 57 each with a prespecified volume of 100,000 cu. ft. (see Fig. 2.6 and Table 2.3). The antecedent moisture condition is fixed as class 2. The results of this study are presented below.

(a) Costs of Drainage Systems With and Without Detention Storage and Their Variation With Design Storm Durations and Frequencies

The variation of drainage system costs with the design storm durations is studied by using six design storm durations (5, 10, 20, 30, 60, and 120 minutes) and three design storm frequencies (2, 5, and 10 years). These three storm frequencies were selected as the study on the Upper Ross-Ade Watershed showed that the lower frequencies are of greater interest and are commonly used. The results are presented in Table 5.3. Similar to the results from the Ross-Ade watershed, in each instance, the cost of the system with detention storages is less than that of a system without them.

The variation of drainage system costs with storm durations is shown in Fig. 5.9. Burke et al. (1980) have examined the hydrologic effects of storm durations on the Bar Barry Heights Subdivision. They concluded that the 20 minute storm duration gave the highest peak runoffs and volumes. The results shown in Fig. 5.9 also indicate that a critical duration of 20 minutes is acceptable for the Bar Barry Heights Subdivision. Two conclusions are obvious from

Table 5.3 Variation of Costs With Storm Duration for Several Return Periods, Bar Barry Heights Subdivision

	2 yrs		5 yrs		10 yrs	
	Design I	Design II	Design I	Design II	Design I	Design II
5 min	576,545	615,927	636,858	694,324	678,845	744,934
10 min	625,988	685,298	729,653	801,818	781,850	887,713
20 min	655,177	719,654	739,094	813,714	821,804	908,757
30 min	621,521	683,457	715,930	792,340	800,858	897,673
60 min	601,531	655,051	644,659	709,193	701,355	775,162
120 min	531,386	569,068	580,754	634,030	604,792	657,278

I: design with a detention storage basin of 100,000 cubic feet each

II: design without detention storage basin

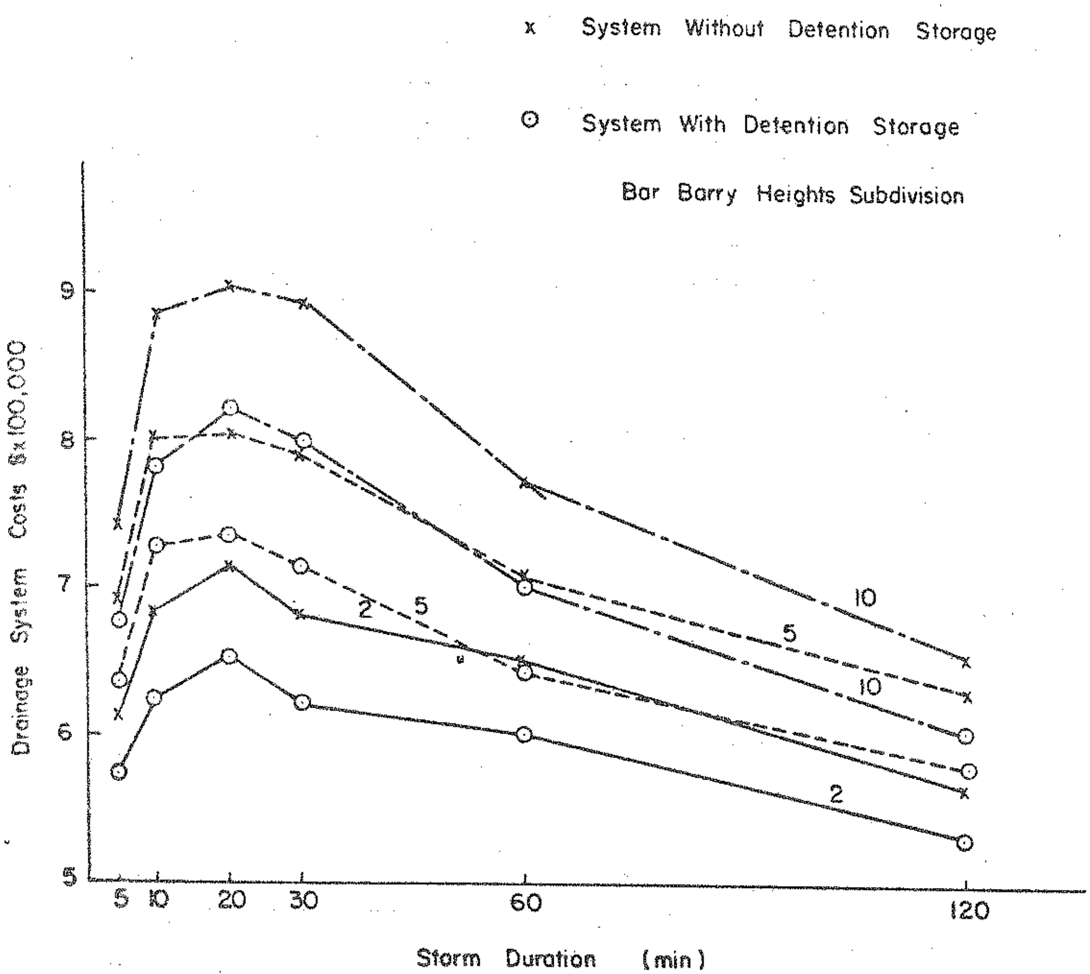

Figure 5.9 Relationships Among Drainage System Costs and Design Storm Duration and Frequencies, Bar Barry Heights Subdivision

the results shown in Fig. 5. 9: (1) the drainage system costs vary considerably with peak runoffs and volumes, and (2) the design storm duration should not be chosen arbitrarily. One method of selecting a proper design storm duration is the method discussed here.

The design storm frequency is one of the most important design parameters in urban drainage design. The American Society of Civil Engineers (1976) reported that a 2-15 year range of frequencies was used in Engineering Offices for storm sewer design in residential areas, with the 5 year frequency as the most common. The relationship between drainage system costs and design storm return periods of critical duration of 20 minutes are presented in Fig. 5.10 for Bar Barry Heights Subdivision. The percent differences in costs of drainage systems designed by using a 10-year design storm or a 2 or 5-year design storm are considerable. Examples of these differences are shown in Table 5.4 for Bar Barry Heights Subdivision. They range from 4% to 12% if a 10-year storm is used instead of a 5-year storm when detention storage is used, and from 4% to 13% when a design without detention storage is used. They range from 14% to 29% if a storm of 10-year frequency is used instead of a storm of a 2-year frequency when a design with detention storages is used, and from 16% to 31% when a design without detention storage is used. On the basis of results of this study, it is recommended that a cost analysis such as that

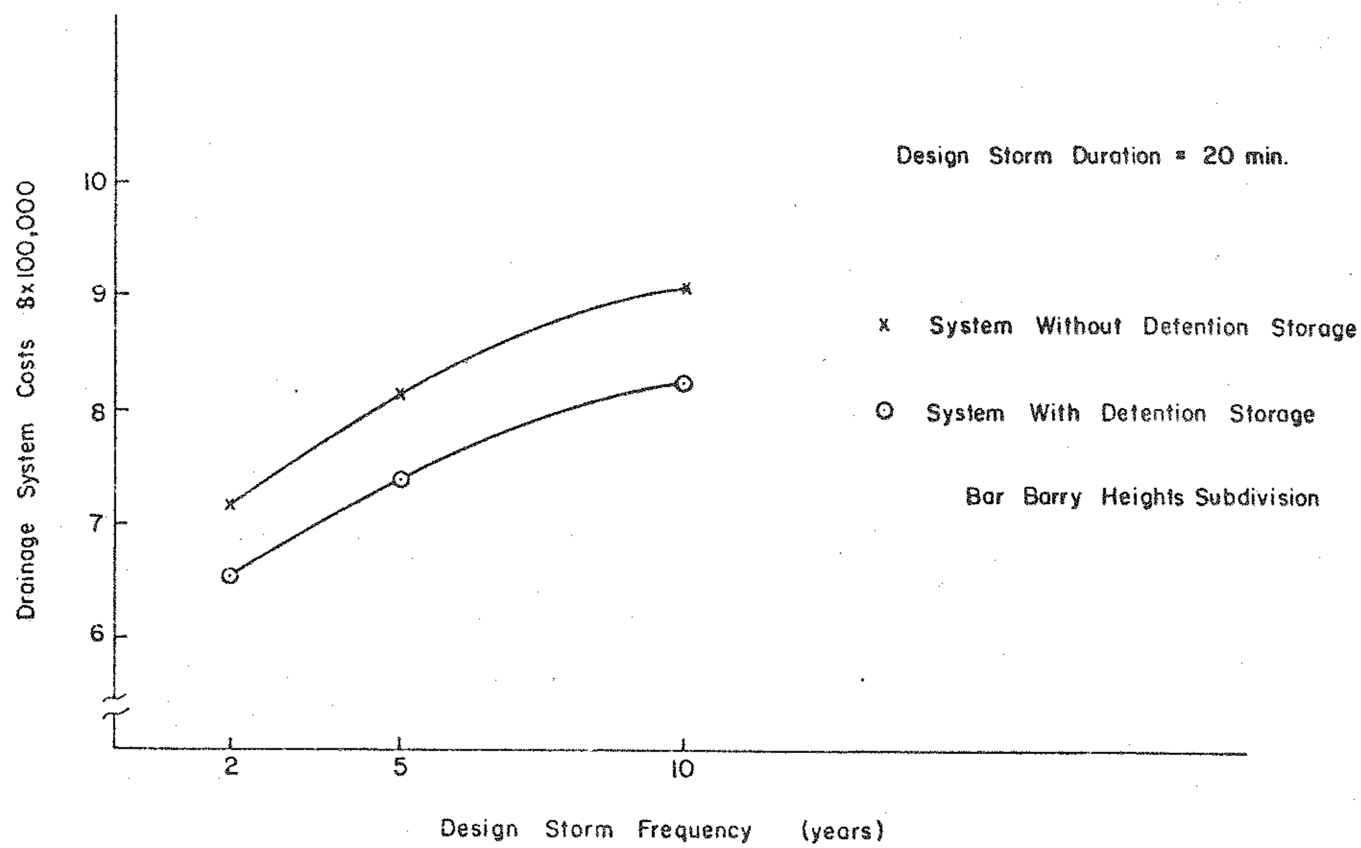

Figure 5.10 Relationships Between Drainage System Costs and Design Storm Frequencies, Bar Barry Heights Subdivision

Table 5.4 Percent Difference In Cost of Drainage Systems Designed by Using Storms of Different Return Periods, Bar Barry Heights Subdivision

	% difference between			
	10 and 5 years		10 and 2 years	
	Design I	Design II	Design I	Design II
5 min	6.6	7.3	17.7	20.9
10 min	7.2	10.7	24.9	29.5
20 min	11.2	11.7	25.4	26.3
30 min	11.9	13.3	28.8	31.3
60 min	8.8	9.3	16.6	18.3
120 min	4.1	3.7	13.8	15.5

I: design with two detention storage basins with 10,000 cubic feet each

II: design without detention storage basin

presented here should be conducted before the final design is accepted.

(b) Variation of Drainage System Costs with AMC

The antecedent moisture condition affects the system costs as it influences the hydrologic response of the watershed. Drainage system costs were computed for different AMC conditions and the critical duration of 20 minutes, and a return period of 5 years. The variation in cost with AMC for both designs, with or without detention storages, is shown in Fig. 5.11. The drainage system design in which AMC 4 is used costs the maximum and provides the most conservative design.

(c) Sensitivity of Drainage System Costs to Input Hyetographs

The sensitivity of costs of drainage systems to input design hyetographs was investigated for Bar Barry Heights Subdivision. The dimensionless mass curves developed from West Lafayette 10-minute rainfall data for the four quartile median storms are used in the study. Costs estimated by using the median curve for the four quartile storms and also the Huff first quartile storm are plotted in Fig. 5.12. These results indicate that the designs based on the default option in ILLUDAS may not always be the most desirable. Consequently dimensionless hyetographs estimated by using local data are preferred.

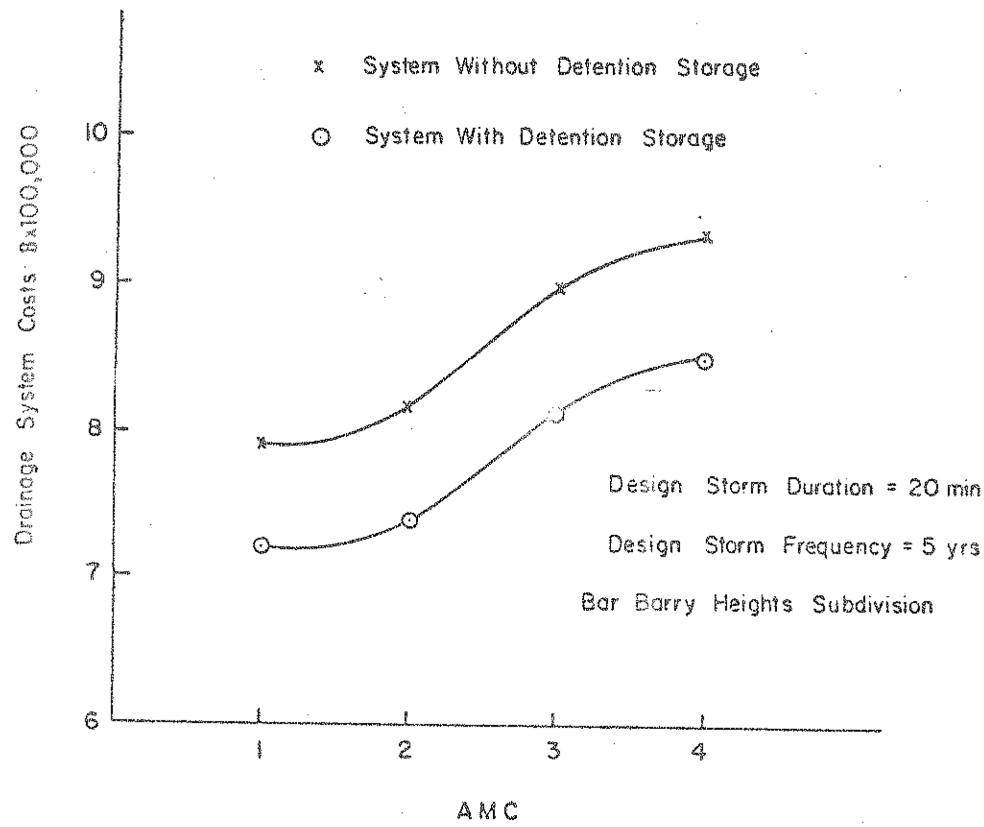

Figure 5.11 Variation in Drainage System Costs With AMC, Bar Barry Heights Subdivision

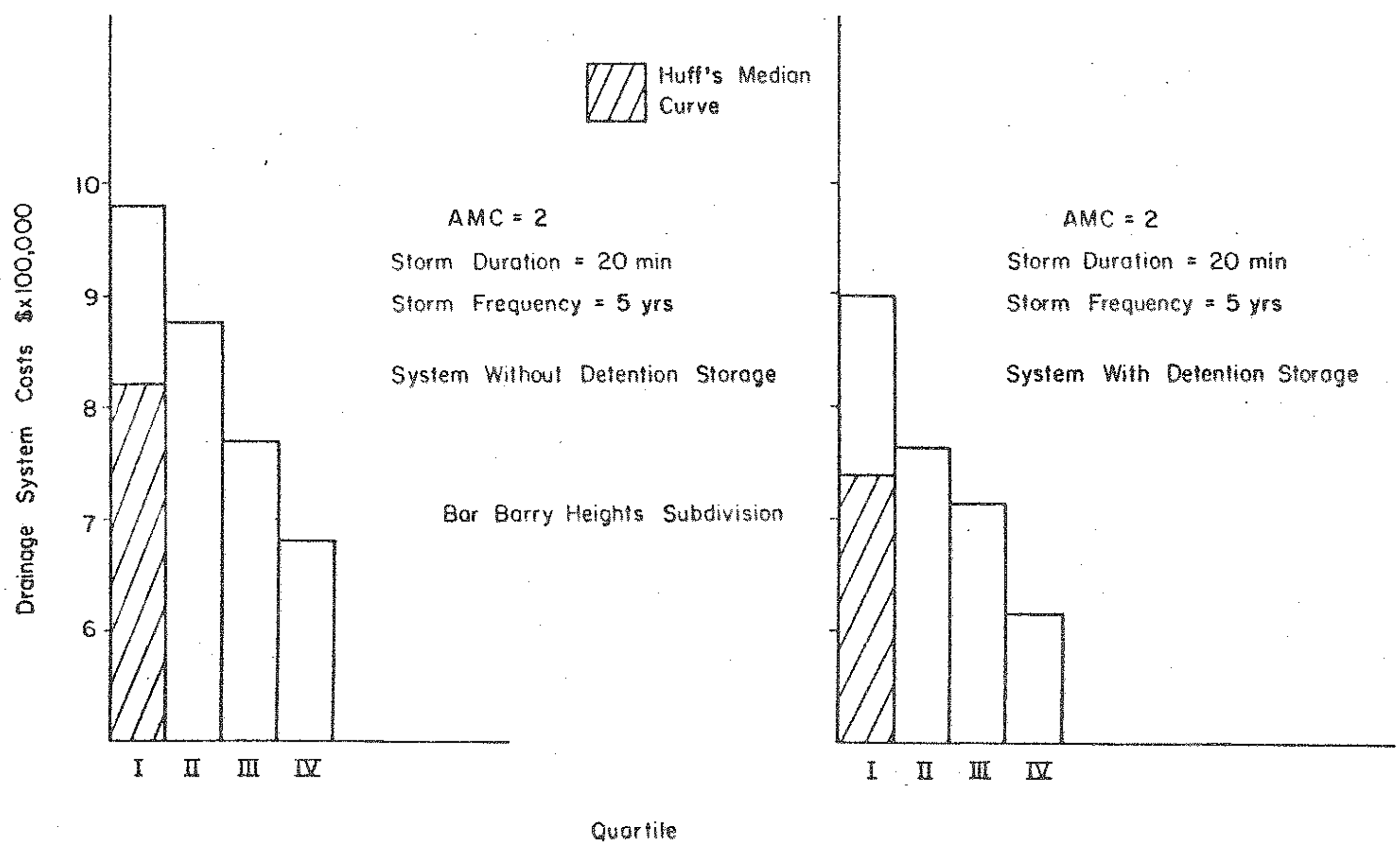

Figure 5. 12 Variation of Costs With the Four Quartile Storms and Huff's Median, First Quartile Storm, Bar Barry Heights Subdivision

(d) Optimal Detention Storage Volumes

The optimal size of a detention storage basin based on costs may be estimated by using the least cost model developed in the present study. The design procedure discussed earlier was used here also. The detention volume and the associated costs are shown in Figs. 5.13 to 5.15 for different design storm return periods and durations. The least cost detention storages are indicated by hollow circles. The complex problem of determining the least cost detention storage basin in the drainage system may be easily solved by using the model developed in the study.

5.3 Summary and Conclusion

A simple, readily usable, and theoretically sound least cost drainage design model is developed and verified by using data from Upper Ross-Ade Watershed and the Bar Barry Heights Subdivision in W. Lafayette, Indiana. The model provides information which would aid designers, engineers, and decision makers involved with urban water management to select least cost optimal designs. The proposed model consists of two major subroutines: the ILLUDAS model and the dynamic programming algorithm. Investigation of the effects of the more important design parameters, the return period, the rainfall duration, the antecedent moisture condition, and the hyetograph on the system costs are investigated. These design parameters significantly affect the system

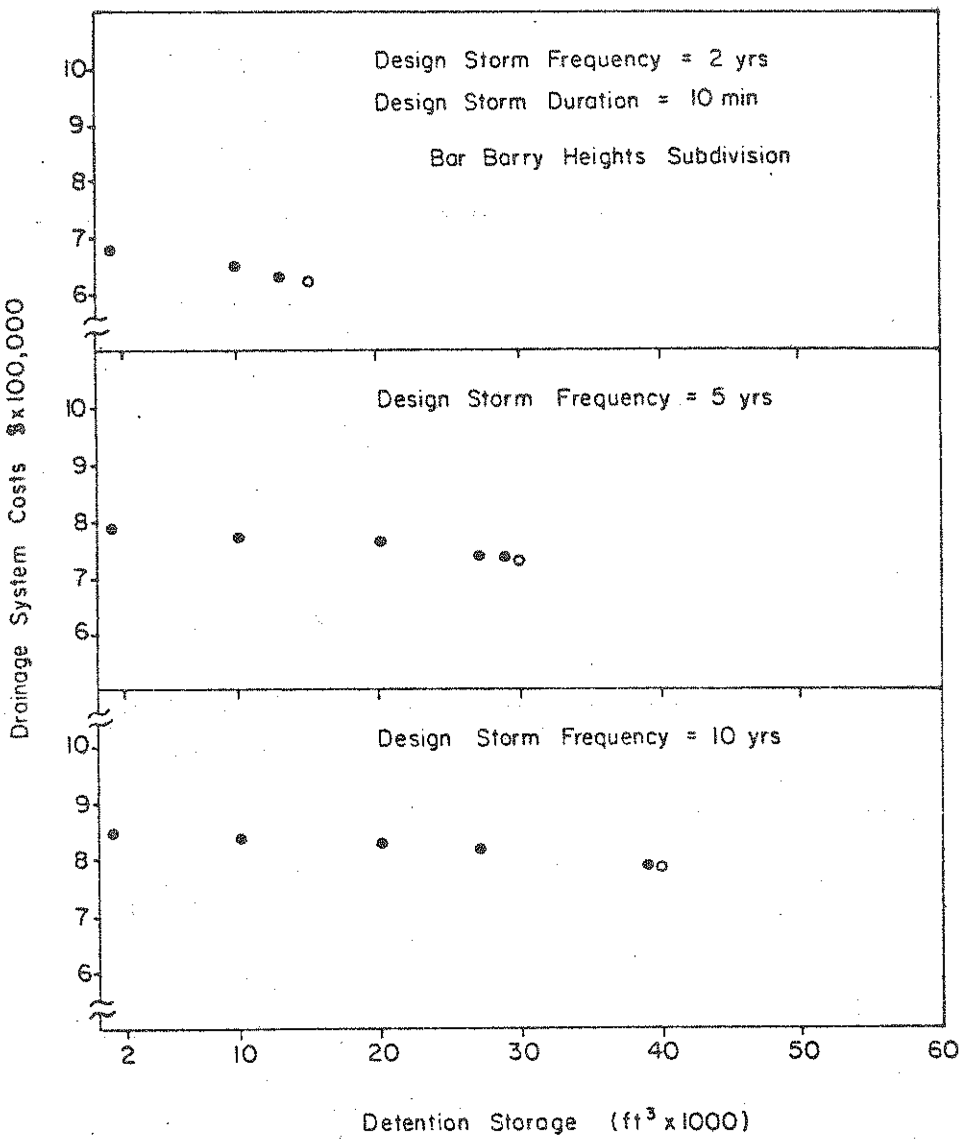

Figure 5.13 Variation of Costs With Detention Storage Volumes, 10 Minutes Storm Duration, Bar Barry Heights Subdivision

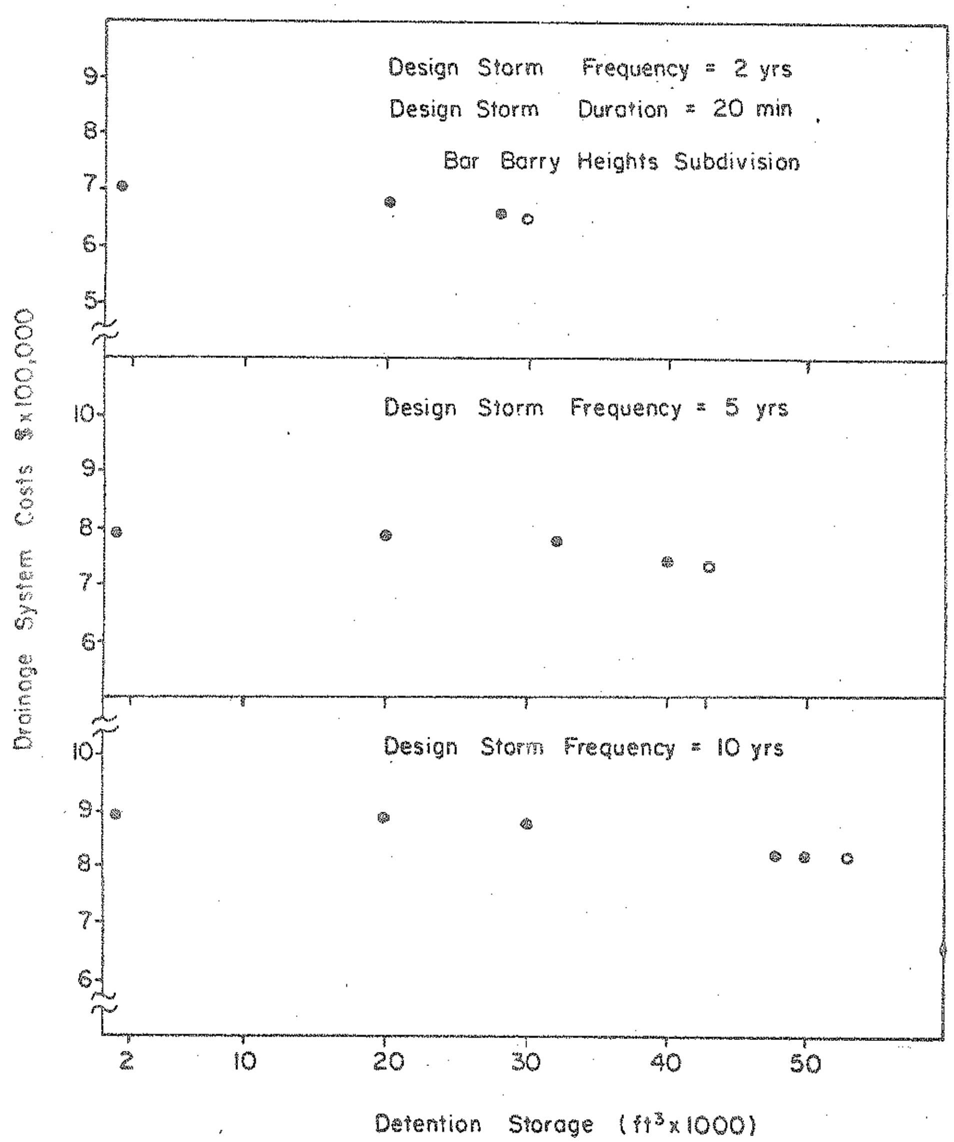

Figure 5.14 Variation of Costs With Detention Storage Volumes, 20 Minutes Storm Duration, Bar Barry Heights Subdivision

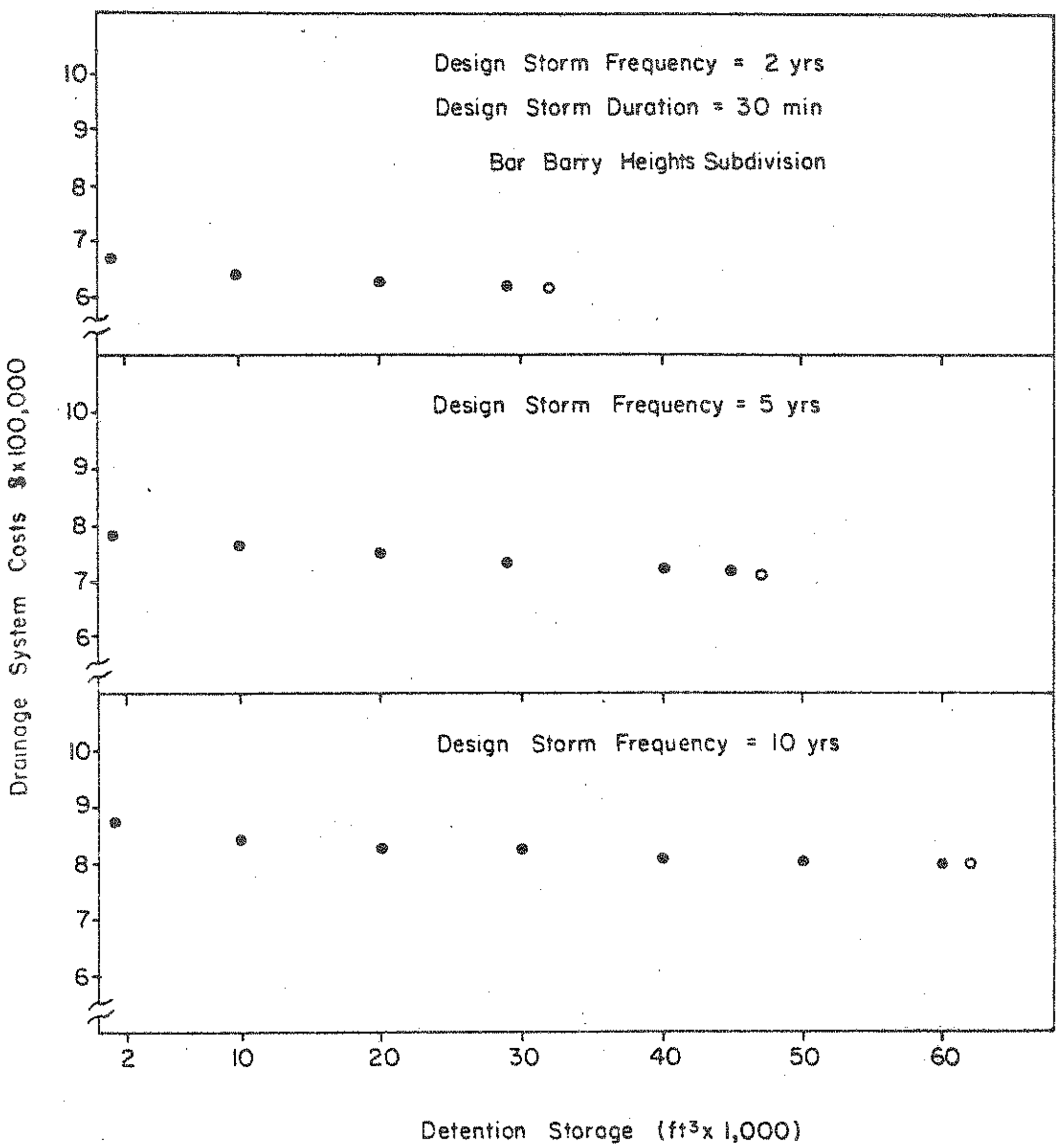

Figure 5.15 Variation of Costs With Detention Storage Volumes, 30 Minutes Storm Duration, Bar Barry Heights Subdivision

costs. A "critical" storm duration, which varies from one system to another exists. For any given drainage system, a critical storm duration or a duration close to it should be used in design. The costs of drainage systems vary more rapidly with the smaller storm return periods (2, 5, and 10 years) than with the higher frequencies (25 and 100 years). The AMC class of 4 is conservative for design. As far as hyetographs used in the design, a design based on the default distribution in ILLUDAS will not be as good as that based on curves derived from data measured at the location where the drainage system is being constructed. The optimal sizes of detention storage basins may be easily determined by using the model.

Finally, it has been demonstrated that the least cost model developed here is easy to apply, practical to use, and yields considerable amounts of information which may be used by engineers. The variation in system costs can be investigated by using this model so that the final design may be selected properly.

A computer program given in the appendix is self contained and can be used to perform the computations discussed in this chapter.

CHAPTER VI

SUMMARY AND CONCLUSIONS

Urban runoff models are required to handle the increasingly complex problems of urban stormwater management. Models of this kind have been developed over the past fifteen years or so. The recognition of the importance of accurate prediction of urban runoff quantity and quality is the main reason for developing these models. Many usable urban runoff models have been developed. However, several questions still remain in using these models for drainage design. One of these is the cost of drainage systems. The optimal least cost design of urban drainage systems is the main problem considered in this study.

The optimal, least cost design of urban drainage systems is of interest for obvious reasons. However, most of the optimal design models assume that the rates of inflow to the drainage system are accurately known. Without a good hydrologic model, these inflow rates cannot be accurately estimated and hence the optimal least cost design model may not be used with great confidence. Consequently, a simple, theoretically sound, and practically usable model which includes ILLUDAS and a dynamic programming subroutine was developed to get least cost drainage system designs.

The model was tested by using data from Upper Ross-Ade Watershed and Bar Barry Heights Subdivision. The model was also used to investigate the variation of system costs with the variation in design parameters such as duration and frequency of rainfall, antecedent moisture condition, and temporal rainfall distribution. For a given watershed, there exists a critical storm duration which produces the highest peak runoff and hence the highest costs. It was found that the system costs vary more for the lower design storm frequencies than for higher frequencies. This is brought out by the percent difference in cost of systems designed by using 10 and 5 years and 10 and 2 years of rainfall. Special attention should be given by the designers to the choice of storm frequency used, as the cost difference may be higher by 50% if 10 year storm is used rather than 2 year storm.

The AMC condition 4 gives the most expensive design and is the most conservative. The hyetograph also affects the cost of drainage systems. It is recommended, wherever possible, that the median quartile storm information derived by an analysis of local data be used. The model may also be used to determine the minimum cost for sizing the detention storage basin. The results obtained are similar for both watersheds.

The least cost drainage design model developed in this study is easy to apply, practical to use, and considerable amounts of information may be easily obtained from the model. The model is helpful to engineers in the design of least cost drainage systems and in the choice of design parameters which are traditionally selected by using "engineering judgment".

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APPPENDIX A

COMPUTER PROGRAM USED IN THE STUDY

The following self contained program is designed to perform the computations required for the least cost design of drainage systems. The method is discussed in Chapter V.

In order to obtain the least cost system the user should call ILLUDAS. The system inflows which are calculated in ILLUDAS are used with subroutine MHHK which is the dynamic programming algorithm to get the least cost system.

The program listing is given below along with a set of input-output data.

```
C     .... ILLUDAS PROGRAM UPDATED OCTOBER, 1979
C     .... FOLLOWING REQUIRED FOR CDC NOT NEEDED  FOR IBM
C
      PROGRAM ILLUDAS (INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
C
C     .... PREVIOUS REQUIRED FOR CDC NOT NEEDED  FOR IBM
C     ....  ILLUDAS -- THE ILLINOIS URBAN DRAINAGE AREA SIMULATOR
C     ....             ILLINOIS STATE WATER SURVEY
C     .... PROGRAM CLEANED UP WITH TIDY PROGRAM APRIL 18,1977
C     .... IF ANY PROBLEMS DEVELOP CONTACT M.L. TERSTRIEP AT ISWS
C     ....          PHONE: 1-217-333-4959
C
      DIMENSION A(100), AR(500), AIF(6), BIF(6), CIF(6), DIF(6), GAD(67)
      DIMENSION GASR(500), GGR(500), GR(500), PQ(10), PV(10), Q(7,501)
      DIMENSION RI(500), RR(500), STORM(20), XNAME(20)
      DIMENSION DIA(24), ALRTE(3)
      COMMON CAPAC,FLAREA,VELOC,A0(51),Q0(51),LSTSCT
      COMMON /HAN/ DESIGNQ(50)
      COMMON /HAN3/ VOL
      COMMON /KELLY/ IQUAR
      REAL KIF
      INTEGER HYD
      DATA AIF(1)/0.0/,AIF(2)/2.0/,AIF(3)/4.0/,AIF(4)/6.0/,AIF(5)/10.0/,
     1AIF(6)/1.0/
      DATA ALRTE/2H  ,2HEX,2HIM/
      DATA BIF(1)/0.0/,BIF(2)/1.5/,BIF(3)/3.0/,BIF(4)/4.0/,BIF(5)/8.0/,B
     1IF(6)/0.5/
      DATA CIF(1)/0.0/,CIF(2)/1.0/,CIF(3)/2.0/,CIF(4)/3.0/,CIF(5)/5.0/,C
     1IF(6)/0.25/
      DATA DIF(1)/0.0/,DIF(2)/0.7/,DIF(3)/1.5/,DIF(4)/2.0/,DIF(5)/3.0/,D
     1IF(6)/0.1/
      DATA END/3HEND/
      DATA IB/1/
      DATA KIF/2.0/
      DATA MAXA/67/
      DATA PQ(1)/.10/,PQ(2)/.20/,PQ(3)/.30/,PQ(4)/.40/,PQ(5)/.50/,PQ(6)/
     1.60/,PQ(7)/.70/,PQ(8)/.80/,PQ(9)/.90/,PQ(10)/1.0/
      DATA PREDI/12.0/
      DATA PV(1)/.16/,PV(2)/.27/,PV(3)/.35/,PV(4)/.43/,PV(5)/.50/,PV(6)/
     1.58/,PV(7)/.65/,PV(8)/.71/,PV(9)/.80/,PV(10)/1.0/
      DATA DIA/8.,10.,12.,15.,18.,21.,24.,27.,30.,36.,42.,48.,54.,60.,72
     1.,84.,96.,108.,120.,132.,144.,156.,168.,180./
      WRITE (6,223)
      LT=5
C
C     IQUAR IS THE SELECTION OF THE DISTRIBUTION OF THE HYETOGRAPH
C     DESIRED. IQUAR=1, BUILT-IN HUFF DISTRIBUTION OF ILLUDAS
C              IQUAR=2, 1ST QUARTILE 50← CURVE OF W. LAF. DATA
C              IQUAR=3, 1ST QUARTILE 10← CURVE OF W. LAF. DATA
C              IQUAR=4, 1ST QUARTILE 90← CURVE OF W. LAF. DATA
C
      IQUAR=1
      LSTSCT=0
      GO TO 102
  101 CONTINUE
      WRITE (6,224)
C
C     .... FOLLOWING FOR CDC SYSTEMS
C
  102 READ (LT,225) XNAME,STORM
C
C     .... PREVIOUS FOR CDC SYSTEMS
C     .... FOLLOWING FOR IBM MACHINES
C     .... 10 READ (LT,1010,END=15) XNAME,STORM
C     .... PREVIOUS FOR IBM MACHINES
C     .... FOLLOWING FOR CDC MACHINES
C
      IF (EOF(LT).NE.0.0) GO TO 103
C
C     .... PREVIOUS FOR CDC MACHINES
```

```
C
      READ (LT,227) XID,DESIN,EVAL,DEBUG,IDXRTE
      IF (IDXRTE.LT.1) IDXRTE=1
      IF (IDXRTE.EQ.1) WRITE (6,217)
      IF (IDXRTE.EQ.2) WRITE (6,218)
      IF (IDXRTE.EQ.2) IDXRTE=3
      IF (IDXRTE.EQ.2.OR.IDXRTE.EQ.3) WRITE (6,219) ALRTE(IDXRTE)
      WRITE (6,226) XNAME,STORM
      READ (LT,228) AREA,ABSTRT,DEPG,ISOIL,DIMIN,RUFFN
C
C     .... PRINT 7, AREA,ABSTRT,DEPG,ISOIL,DIMIN,RUFFN
C
      READ (LT,229) RAIN,XRI,DELT,HUFF,DURA,FREQ,TRAIN,AMC
C
C     .... PRINT 21,RAIN,XRI,DELT,HUFF,DURA,FREQ,TRAIN,AMC
C
      IF (DESIN.NE.0.0.AND.EVAL.NE.0.0) GO TO 104
      IF (DESIN.EQ.0.0.AND.EVAL.EQ.0.0) GO TO 107
      IF (DESIN.EQ.0.0) GO TO 106
      GO TO 105
  103 CONTINUE
      WRITE (6,230)
      STOP
  104 WRITE (6,231)
  105 IRUNB=1
      GO TO 108
  106 IRUNB=2
      GO TO 108
  107 WRITE (6,232)
      IRUNB=1
  108 CONTINUE
      NRI=XRI
      IFREQ=FREQ
      IMC=AMC
      IID=XID
      IF (HUFF.GT.0.0.AND.RAIN.GT.0.0) GO TO 110
      IF (RAIN.EQ.0.0) GO TO 111
      READ (LT,233) (RR(J),J=1,NRI)
      TRAIN=0
      DO 109 K=1,NRI
         TRAIN=TRAIN+RR(K)
  109 CONTINUE
      XNRI=NRI
      DURA=(XNRI-1.0)*DELT
      GO TO 112
  110 WRITE (6,234)
      GO TO 216
  111 CONTINUE
      CALL RHUFF (TRAIN,DURA,DELT,RR,NRI)
  112 CONTINUE
      WRITE (6,235)
      WRITE (6,236) (RR(J),J=1,NRI)
      WRITE (6,237)
      WRITE (6,238)
      WRITE (6,239) IID,AREA,DELT,ISOIL
      WRITE (6,240)
      WRITE (6,241)
      WRITE (6,242) TRAIN,IFREQ,DURA,IMC,ABSTRT,DEPG
      WRITE (6,243)
      WRITE (6,244)
      PREDI=DIMIN
      DELTAT=DELT/60.0
      NEND=0
      DO 113 L=1,500
         GR(L)=0.0
  113 CONTINUE
      DO 114 M=1,6
         Q(M,501)=0.0
  114 CONTINUE
      TGAR=0.0
```

```
      TGA=0.0
      TSPA=0.0
      TPAR=0.0
      TCPA=0.0
      ILL=0
      MNHOLE=0
  115 CONTINUE
      MNHOLE=MNHOLE+1
      VOL=0
      OUTLET=0
      SURMAX=0
      SMX=0
      READ (LT,245) BRAN,REACH,ENDBR,CONBR,IRUN,DIST,SLP,RUFF,ISECT,DIAM
     1,HR,WR,SS,QALOW,FREQR,STORE,TEST,HYD
      IF (DEBUG.GT.0.0) HYD=1
      IF (IRUN.NE.0) GO TO 116
      IRUN=IRUNB
  116 CONTINUE
      IF (ENDBR.NE.0.0) GO TO 206
      READ (LT,246) CBRAN,CREACH,BA,CPA,PCPA,SPA,PSPA,PENT,PL,PS,CGA,PCG
     1A,GENT,GL,GS,IGROUP
C
C     .... PRINT 6, BRAN,REACH,ENDBR,CONBR,DIST,SLP,RUFF,ISECT,DIAM,HR,W
C     .... 1SS,FREQR,STORE,HYD
C     .... PRINT 5, CBRAN,CREACH,BA,CPA,PCPA,SPA,PSPA,PENT,PL,PS,CGA,PCG
C     .... 1GENT,GL,GS,IGROUP
C
      WRITE (6,272) BRAN,REACH
      IF (SLP) 117,117,118
  117 WRITE (6,273) BRAN,REACH
      WRITE (6,274)
      STOP
  118 CALL EXIST (ISECT,DIAM,RUFF,SLP,HR,WR,SS,ECAP,EVEL)
      IF (IGROUP.EQ.0) IGROUP=ISOIL
      IF (FREQR.EQ.1.0) GO TO 121
      IF (FREQR.NE.0.0) GO TO 119
      FREQR=1.0
      GO TO 121
  119 DO 120 IJ=1,NRI
         RR(IJ)=RR(IJ)*FREQR
  120 CONTINUE
      WRITE (6,247) FREQR
  121 CONTINUE
      IF (CPA.NE.0.0) GO TO 122
      CPA=BA*PCPA*0.01
  122 IF (SPA.NE.0.0) GO TO 123
      SPA=BA*PSPA*0.01
  123 IF (CGA.NE.0.0) GO TO 124
      CGA=BA*PCGA*0.01
  124 IF (PENT+PL.EQ.0.0) GO TO 125
      IF (PENT.NE.0.0) GO TO 125
      IF (CPA.EQ.0.0) GO TO 125
      CALL PAVENT (PENT,PL,PS,CPA)
  125 CONTINUE
      TGA=TGA+CGA
      TSPA=TSPA+SPA
      TCPA=TCPA+CPA
      WRITE (6,248) TCPA,TSPA,TGA
      IF (BRAN.EQ.0.0) GO TO 126
      IF (CPA+CGA+SPA) 161,161,128
  126 CONTINUE
      IF (ENDBR.EQ.0.0) GO TO 127
C
C     .... LABEL 600 IS FOR A CONFLUENCE
C
      GO TO 206
  127 WRITE (6,249)
      GO TO 215
  128 IF (CPA) 129,129,131
  129 DO 130 N=1,500
```

```
         GR(N)=0.0
  130 CONTINUE
      GO TO 138
  131 CALL TIMEA (A,PENT,DELT,NAI,MAXA,CPA)
      DO 132 N=1,NRI
         RI(N)=RR(N)
  132 CONTINUE
      CALL INTEN (RI,ABSTRT,NRI,DELTAT)
C
C     .... COMPUTE GROSS PAVED AREA HYDROGRAPH
C
      NEND=NRI+NAI-1
      DO 133 J=1,500
  133 GR(J)=0
      DO 135 L=1,NRI
         N=L-1
         DO 134 J=1,NAI
            N=N+1
            DGR=RI(L)*A(J)
            GR(N)=GR(N)+DGR
  134    CONTINUE
  135 CONTINUE
      IF (HYD) 137,137,136
  136 WRITE (6,250) BRAN,REACH
      WRITE (6,251) (GR(J),J=1,NEND)
  137 IF (CGA) 157,157,138
  138 CONTINUE
      GO TO (139,140,141,142), IGROUP
  139 FI=AIF(IMC)
      FO=AIF(5)
      FC=AIF(6)
      GO TO 143
  140 FI=BIF(IMC)
      FO=BIF(5)
      FC=BIF(6)
      GO TO 143
  141 FI=CIF(IMC)
      FO=CIF(5)
      FC=CIF(6)
      GO TO 143
  142 FI=DIF(IMC)
      FO=DIF(5)
      FC=DIF(6)
  143 CONTINUE
C
C     .... PRINT 406, NRI,IGROUP,IMC,CGA,SPA,DELTAT,DEPG,GL,GS,FI,FO,FC
C     .... PRINT 407,(RR(J),J=1,NRI)
C
      DO 144 I=1,NRI,1
         AR(I)=RR(I)*(CGA+SPA)/CGA
  144 CONTINUE
      CALL SUPPLY (AR,DELTAT,FC,FI,FO,GASR,KIF,NRI,DEPG,NGSR,SGASR)
      IF (NGSR) 145,145,146
  145 IF (CPA.GT.0.0) GO TO 157
      GO TO 161
  146 CONTINUE
C
C     .... PRINT 461, (GASR(I),I=1,NGSR,1)
C     .... PRINT 462, SGASR
C
      IF (GENT+GL.EQ.0.0) GO TO 147
      IF (GENT.NE.0.0) GENT=GENT+PENT
      IF (GENT.NE.0.0) GO TO 148
      CALL GRENT (GENT,CGA,GL,GS,PENT)
      GO TO 148
  147 GENT=20.0
      WRITE (6,252)
      GO TO 148
  148 CONTINUE
      CALL TIMEA (GAD,GENT,DELT,NGAI,MAXA,CGA)
```

```
      NGEND=NGAI+NGSR-1
      DO 149 J=1,500
  149 GGR(J)=0.0
      DO 151 L=1,NGSR
         N=L-1
         DO 150 J=1,NGAI
            N=N+1
            GDGR=GASR(L)*GAD(J)
            GGR(N)=GGR(N)+GDGR
  150    CONTINUE
  151 CONTINUE
      IF (HYD) 153,153,152
  152 WRITE (6,253)
      WRITE (6,254) (GGR(J),J=1,NGEND)
  153 IF (NGEND-NEND) 155,155,154
  154 NEND=NGEND
  155 DO 156 I=1,NEND
         GR(I)=GR(I)+GGR(I)
  156 CONTINUE
  157 CONTINUE
C
C     .... PRINT 59,CPA,CGA,SPA,PENT,GENT
C
      GRPK=GR(1)
      DO 159 J=2,NEND
         IF (GRPK-GR(J)) 158,159,159
  158    GRPK=GR(J)
  159 CONTINUE
      PKIN=GRPK
      LAST=NEND
C
C     .... TEST FOR MID BRANCH (421)  OR INITIAL (426)
C
  160 IF (REACH.NE.0.0) GO TO 163
      GO TO 173
C
C     .... FOR AREA =0 IN MID BRANCH
C
  161 DO 162 J=1,500
         GR(J)=0.0
  162 CONTINUE
      GRPK=0.0
      PKIN=0.0
      GO TO 160
C
C     .... COMBINE PREVIOUS ROUTED HYDROGRAPH WITH NEW GROSS HYDROGRAPH
C
  163 CONTINUE
      DO 164 M=1,6
         IF (Q(M,501).EQ.BRAN) GO TO 165
  164 CONTINUE
      WRITE (6,255)
      GO TO 215
  165 IB=M
      GRPK=0
      DO 168 N=1,500
         GR(N)=GR(N)+Q(IB,N)
         IF (GRPK-GR(N)) 166,167,167
  166    GRPK=GR(N)
  167    CONTINUE
  168 CONTINUE
      IF (HYD) 170,170,169
  169 WRITE (6,256)
      WRITE (6,257) (GR(J),J=1,LAST)
  170 IF (DIAM) 171,171,172
  171 TDIAM=PREDI
C
C     .... LABEL 450 IS FOR ROUTING
C
      GO TO 182
```

```
  172 TDIAM=DIAM
      IF (IRUN.EQ.1) TDIAM=DIMIN
      GO TO 182
  173 CONTINUE
      IF (DIAM) 178,178,174
  174 TDIAM=DIAM
      IF (IRUN.EQ.1.AND.DIMIN.LT.8.0) WRITE (6,220)
      IF (IRUN.EQ.1.AND.DIMIN.LT.8.0) DIMIN=8.0
      FLG1=0.0
      DO 175 I=1,22
  175 IF (DIMIN.EQ.DIA(I)) FLG1=1.0
      IF (FLG1.EQ.1.0) GO TO 177
      DIFF=1000.0
      ICHOOS=1
      DO 176 I=1,24
         IF (ABS(DIMIN-DIA(I)).GT.DIFF) GO TO 176
         DIFF=ABS(DIMIN-DIA(I))
         ICHOOS=I
  176 CONTINUE
      DIMIN=DIA(ICHOOS)
      IF (FLG1.EQ.1.0) WRITE (6,221) DIMIN
  177 CONTINUE
      GO TO 179
  178 TDIAM=DIMIN
  179 DO 180 M=1,6
         IF (Q(M,501).EQ.0.0) GO TO 181
  180 CONTINUE
      WRITE (6,258)
      GO TO 215
  181 IB=M
      Q(IB,501)=BRAN
      GO TO 182
C
C     .... FIND GROSS HYDROGRAPH PEAK
C
  182 GRPK=GR(1)
      DO 184 J=2,500
         IF (GRPK-GR(J)) 183,184,184
  183    GRPK=GR(J)
  184 CONTINUE
      PKDES=GRPK
      IF (STORE.EQ.0.0) GO TO 185
      IF (QALOW.EQ.0.0) GO TO 187
      WRITE (6,259)
      QALOW=0.0
      GO TO 187
  185 IF (QALOW.EQ.0.0) GO TO 189
      IF (QALOW-GRPK) 186,189,189
  186 CALL LIMITQ (GR,GRPK,LAST,QALOW,DELT,BRAN,REACH,VOL)
      GO TO 188
  187 CALL DETEN (GR,GRPK,LAST,STORE,DELT,BRAN,REACH,VOL)
  188 CONTINUE
      OUTLET=GRPK
      SMX=STORE*1000.0
C
C     .... PRINT 903,BRAN,REACH,SMX,GRPK,VOL
C
  189 GO TO (190,194,194), IRUN
  190 CONTINUE
      INCR=0
  191 QFB=0.0081*TDIAM*TDIAM/RUFFN*(TDIAM/48.0)**.667*(SLP/100.0)**.50
      VFB=QFB/(TDIAM*TDIAM*3.141592654/576.)
      IF (QFB-GRPK) 192,193,193
  192 CONTINUE
      IF (INCR.GT.24) WRITE (6,222)
      IF (INCR.GT.24) STOP
      INCR=INCR+1
      TDIAM=DIA(INCR)
      GO TO 191
  193 CONTINUE
```

```
  194 CALL ROUTE (GR,IB,DELT,RUFF,SLP,DIAM,DIST,LAST,SURMAX,Q,VOL,HYD,IS
     1ECT,HR,WR,SS,GRPK,BRAN,REACH,IDXRTE,IRUN,RUFFN,TDIAM,QFB)
      IF (QALOW.NE.0.0.OR.STORE.NE.0.0) GO TO 195
      GO TO (196,197,197), IRUN
  195 GO TO (198,199,199), IRUN
  196 WRITE (6,260) BRAN,REACH,DIST,SLP,RUFFN,HR,WR,SS,DIAM,ECAP,EVEL,OU
     1TLET,VOL,SMX
      WRITE (6,261) TDIAM,QFB,VFB,PKDES,PKIN,SURMAX
      DESIGNQ(MNHOLE)=PKDES
      IF (DIAM.GE.TDIAM) WRITE (6,262)
      GO TO 200
  197 WRITE (6,260) BRAN,REACH,DIST,SLP,RUFF,HR,WR,SS,DIAM,ECAP,EVEL,OUT
     1LET,VOL,SMX
      WRITE (6,263) PKDES,PKIN,SURMAX
      GO TO 200
  198 WRITE (6,264) BRAN,REACH,DIST,SLP,RUFFN,HR,WR,SS,DIAM,ECAP,EVEL,PK
     1DES,PKIN,SURMAX
      WRITE (6,265) TDIAM,QFB,VFB,OUTLET,VOL,SMX
      DESIGNQ(MNHOLE)=OUTLET
      GO TO 200
  199 WRITE (6,264) BRAN,REACH,DIST,SLP,RUFF,HR,WR,SS,DIAM,ECAP,EVEL,PKD
     1ES,PKIN,SURMAX
      WRITE (6,266) OUTLET,VOL,SMX
  200 CONTINUE
C
C     .... PRINT 1301,BRAN,REACH,ISECT,DIAM,HR,WR,SLP,RUFF,DEPTH,SURMAX
C     .... FIND PEAK OF DISCHARGE HYDROGRAPH
C
      QPK=0
      DO 202 ID=1,LAST
         IF (Q(IB,ID)-QPK) 202,202,201
  201    QPK=Q(IB,ID)
  202 CONTINUE
C
      PREDI=TDIAM
      IRUN=IRUNB
      IF (FREQR.EQ.1.0) GO TO 204
      DO 203 IJ=1,NRI
         RR(IJ)=RR(IJ)/FREQR
  203 CONTINUE
  204 CONTINUE
      IF (TEST.NE.END) GO TO 115
C
C     .... PRINT DISCHARGE HYDRO
C     .... WRITE (6,402)
C     .... WRITE(6,403)PENT,RUFF,DELT,FREQR
C
      VOLOUT=0.0
      DO 205 M=1,LAST
         VOLOUT=VOLOUT+Q(IB,M)
  205 CONTINUE
      VOLOUT=VOLOUT*DELT*60.
      WRITE (6,267) VOLOUT
      WRITE (6,268) (Q(IB,M),M=1,LAST)
      CALL MHHK
      GO TO 101
C
C     .... PRINT RESULTS FOR NEWLY DESIGNED REACH
C     .... COMBINE ROUTED HYDROS AT A CONFLUENCE
C
  206 CONTINUE
      MNHOLE=MNHOLE-1
      DO 207 M=1,6
         IF (Q(M,501).EQ.CONBR) GO TO 208
  207 CONTINUE
      WRITE (6,269)
      GO TO 215
  208 IB=M
      DO 209 M=1,6
         IF (Q(M,501).EQ.ENDBR) GO TO 210
```

```
  209 CONTINUE
      WRITE (6,270)
      GO TO 215
  210 IEND=M
      DO 211 N=1,500
         Q(IB,N)=Q(IB,N)+Q(IEND,N)
  211 CONTINUE
C
C     .... PRINT 1326
C     .... PRINT 1327,(Q(IB,J,)J=1,200)
C
      Q(IEND,501)=0.0
      GO TO 212
  212 LAST=1
      DO 214 N=1,500
         IF (Q(IB,N).GT.0.0) GO TO 213
         GO TO 214
  213    LAST=N
  214 CONTINUE
      IF (TEST.EQ.END) GO TO 101
      GO TO 115
  215 WRITE (6,271)
  216 CONTINUE
C
C
  217 FORMAT (32H   TIME SHIFT ROUTING ACTIVATED.)
  218 FORMAT (46H **** EXPLICIT ROUTING NO LONGER ALLOWED **** )
  219 FORMAT (3X,A2,36HPLICIT HYDROLOGIC ROUTING ACTIVATED.)
  220 FORMAT (45H MINIMUM PIPE DIA SPECIFIED ) 8-IN, 8-IN USED)
  221 FORMAT (51H CLOSEST COMMERCIALLY AVAILABLE PIPE SIZE CHOSEN AS,F10
     1.2,8H INCHES.)
  222 FORMAT (49H PIPE SIZING ← 180-IN: PROBABLE INPUT DATA ERROR.)
  223 FORMAT (47H1     ILLUDAS ** ILLINOIS STATE WATER SURVEY ** ,/,59H
     1 ILLUDAS UPDATED  OCT,  1979  WITH COMMERCIAL PIPE SIZING)
  224 FORMAT (1H1)
  225 FORMAT (20A4)
  226 FORMAT (///,20A4,/,20A4,//)
  227 FORMAT (4F10.0,I10)
  228 FORMAT (3F10.0,I10,2F10.0)
  229 FORMAT (8F10.0)
  230 FORMAT (21H  THE JOB IS FINISHED)
  231 FORMAT (54H DESIGN AND EVALUATION BOTH SPECIFIED - DESIGN ASSUMED)
  232 FORMAT (52H NEITHER DESIGN NOR EVAL SPECIFIED - DESIGN ASSUMED )
  233 FORMAT (10F8.0)
  234 FORMAT (48H RAINFALL PROVIDED OR STANDARD DISTRIBUTION &&& )
  235 FORMAT (18H RAINFALL PATTERN )
  236 FORMAT (10F8.3)
  237 FORMAT (//,59H       RUN NUMBER   BASIN AREA   TIME INCREMENT   SOI
     1L GROUP)
  238 FORMAT (59H                          ACRES         MINUTES      1234=A
     1BCD ,/)
  239 FORMAT (I13,F15.1,F13.1,I13,//)
  240 FORMAT (73H        TOTAL RAIN   FREQUENCY   DURATION    AMC   PAVED
     1 ABS.   GRASS ABS.)
  241 FORMAT (70H          INCHES        YEARS      MINUTES          INCHES
     1        INCHES,/)
  242 FORMAT (9X,F5.2,6X,I5,7X,F6.1,I8,F11.2,F14.2,//)
  243 FORMAT (//,107H      B    R    LENG  SLP     N    HT    BW  V/H   DIA
     1 CAPAC    VEL     DESIGN     INLET     DETENTION     STORAGE )
  244 FORMAT (108H                     FT    PCT          FT    FT          INS     C
     1FS     FPS      Q-CFS      Q-CFS      CUBIC FT   REQUESTED ,//)
  245 FORMAT (4F3.0,I3,3F5.0,I1,F4.0,6F5.0,1A3,I2,10X)
  246 FORMAT (2F3.0,F9.0,F5.0,F3.0,F5.0,F3.0,4F5.0,F3.0,3F5.0,I2,9X)
  247 FORMAT (36H RAINFALL MULTIPLIED BY A FACTOR OF ,F5.2,15H FOR THIS
     1REACH)
  248 FORMAT (5X,24HACCUM CONTRIBUTING AREAS,7H   CPA=,F7.1,8H,    SPA=,F
     17.1,8H,    CGA=,F7.1)
  249 FORMAT (34H  BRANCH AND ENDBR BOTH EQUAL ZERO)
  250 FORMAT (22H PAVED AREA HYDROGRAPH,2F10.1)
  251 FORMAT (10F8.1)
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252 FORMAT (45H GRASS ENT ASSUMED = 20 MIN.  GIVE MORE DATA )
253 FORMAT (24H GRASSED AREA HYDROGRAPH)
254 FORMAT (10F8.1)
255 FORMAT (37H PREVIOUS BRANCH HYDROGRAPH NOT FOUND)
256 FORMAT (41H UPSTREAM ROUTED PLUS SURFACE HYDROGRAPH.)
257 FORMAT (10F8.1)
258 FORMAT (21H NO BRANCHES ARE FREE)
259 FORMAT (53H BOTH STORAGE AND LIMITED Q REQUESTED - STORAGE USED )
260 FORMAT (F8.0,F4.0,F6.0,F5.2,F6.3,3F5.2,F5.0,F8.2,F6.2,F10.2,9X,F12
   1.1,F13.0,/)
261 FORMAT (44H                    REQUIRED PIPE =          ,F5.0,F8.2,
   1F6.2,F10.2,F9.2,F13.2,//)
262 FORMAT (10X,41H***EXISTING PIPE HAS ADEQUATE CAPACITY***//)
263 FORMAT (63H
   1        ,F10.2,F9.2,F13.2,//)
264 FORMAT (F8.0,F4.0,F6.0,F5.2,F6.3,3F5.2,F5.0,F8.2,F6.2,F10.2,F9.2,F
   113.2,/)
265 FORMAT (44H                    REQUIRED PIPE =          ,F5.0,F8.2,
   1F6.2,F10.2,9X,F12.1,F13.0,//)
266 FORMAT (63H
   1        ,F10.2,9X,F12.1,F13.0,//)
267 FORMAT (///,58H   OUTFALL HYDROGRAPH IN CFS, ACCUMULATED RUNOFF IN
   1 CU FT=,F12.0)
268 FORMAT (10F8.1)
269 FORMAT (35H CONTINUING BRANCH RECORD NOT FOUND)
270 FORMAT (29H ENDD BRANCH RECORD NOT FOUND)
271 FORMAT (38H  TROUBLE FINDING UPSTREAM HYDROGRAPH )
272 FORMAT (///,20H  ****  BEGIN BRANCH,F5.0,7H REACH ,F5.0,6H **** ,/
   1//)
273 FORMAT (1H0,22H **** ERROR IN BRANCH ,F5.0,7H REACH ,F5.0,6H ****
   1)
274 FORMAT (1H0,46H **** EXECUTION TERMINATED BY ZERO SLOPE **** )
C
      END
      SUBROUTINE SUPPLY (AR,DELTAT,FC,FI,FO,GASR,K,NRI,DEPG,NGSR,SGASR)
      DIMENSION GASR(500), AR(500)
      REAL K,IS
C
C     .... PRINT 105, DELTAT,FC,FI,FO,K,NRI,DEPG
C     .... PRINT 4,(AR(I),I=1,NRI)
C
      SGASR=0.0
      F1=FI
      MK=1
      AS=DEPG
      IS=0
      DO 112 I=1,NRI,1
         IF (MK) 104,104,101
 101     T=0.0
         TT=0.0
 102     CONTINUE
         F=FC*T+((1.-EXP(-K*T))*(FO-FC))/K
         F=F-F1
         FP=FC+((FO-FC)*(K*EXP(-K*T)))/K
         T=T-F/FP
         IF (ABS(TT-T).LT.0.001) GO TO 103
         TT=T
         GO TO 102
 103     CONTINUE
 104     TN=T+DELTAT
         FN=FC*TN+((1.-EXP(-K*TN))*(FO-FC))/K
         FINC=FN-F1
         RINC=AR(I)
         DRUN=RINC-FINC
C
C     ....                            PRINT 5,FO,FC,F1,T,TN,FN,AS,DRUN
C
         IF (DRUN) 105,108,109
 105     IS=DEPG-AS
         IF (ABS(DRUN)-IS) 107,106,106
```

```
  106      CONTINUE
           AS=DEPG
           GASR(I)=0.0
           F1=F1+RINC+IS
           IS=0
           MK=1
           GO TO 112
  107      IS=IS+DRUN
           AS=DEPG-IS
           GASR(I)=0.0
           F1=FN
           T=TN
           MK=-1
           GO TO 112
  108      F1=FN
           T=TN
           MK=-1
           GASR(I)=0.0
           GO TO 112
  109      F1=FN
           T=TN
           MK=-1
           IF (DRUN-AS) 111,110,110
  110      GASR(I)=DRUN-AS
           AS=0.0
           GO TO 112
  111      AS=AS-DRUN
           GASR(I)=0.0
           GO TO 112
  112 CONTINUE
      J=NRI+1
      DO 113 I=J,500,1
         GASR(I)=0.0
  113 CONTINUE
      NGSR=0
      DO 115 J=1,NRI
         IF (GASR(J).LT.0.001) GO TO 114
         NGSR=J
         SGASR=SGASR+GASR(J)
         GASR(J)=GASR(J)/DELTAT
         GO TO 115
  114    GASR(J)=0.0
  115 CONTINUE
C
C     .... PRINT 102
C     .... PRINT 103,(GASR(I),I=1,NRI)
C
      RETURN
C
      END
      SUBROUTINE TIMEA (A,ENT,DELT,NAI,MAXA,CA)
      DIMENSION A(67)
C
C     .... COMPUTE AND STORE TIME AREA CURVE
C
      AAS=ENT/DELT
      TAAS=AAS+1.0
      NAI=TAAS
      IF (NAI.EQ.1) GO TO 103
      ASUM=0
      NIX=NAI-1
      DO 101 N=1,NIX
         A(N)=CA/AAS
  101 ASUM=ASUM+A(N)
      A(NAI)=CA-ASUM
      NAX=NAI+1
      DO 102 N=NAX,MAXA
  102 A(N)=0
      GO TO 105
  103 A(1)=CA
```

```
      DO 104 N=2,MAXA
  104 A(N)=0
  105 CONTINUE
C
C     .... PRINT 70
C     .... PRINT 80,(A(N),N=1,NAI)
C
      RETURN
C
      END
      SUBROUTINE GRENT (GENT,GA,GLENG,GSLP,ENT)
      DATA AVSP/1.0/,C/0.050/
C
C     ....    DETERMINE GRASSED AREA ENTRY TIME BY IZZARD EQUATIONS
C
      QEQ=AVSP*GLENG/43200.
      CK=(0.0007*AVSP+C)/(GSLP/100.0)**0.333
      DET=CK*GLENG*QEQ**0.4
      GGENT=DET/(30.0*QEQ)
      GENT=GGENT+ENT
      WRITE (6,101) GENT
      RETURN
C
C
  101 FORMAT (21H GRASSED ENTRY TIME= ,F6.1,4H MIN)
C
      END
      SUBROUTINE INTEN (RI,ABSTRT,NRI,DELTAT)
      DIMENSION RI(500)
      SUB=0.0
      DO 101 J=1,NRI
         SUB=SUB+RI(J)
         IF (ABSTRT-SUB) 102,102,101
  101 RI(J)=0.0
      WRITE (6,105)
      GO TO 104
  102 CONTINUE
      RI(J)=SUB-ABSTRT
      DO 103 K=1,NRI
         RI(K)=RI(K)/DELTAT
  103 CONTINUE
  104 CONTINUE
C
C     .... PRINT 70,(RI(J),J=1,NRI)
C
      RETURN
C
C
  105 FORMAT (51H  ABSTRAT GREATER THAN RAINFALL IN SUBROUTINE INTEN)
C
      END
      SUBROUTINE DETEN (GR,GRPK,LAST,STORE,DELT,BRAN,REACH,VOL)
      DIMENSION GR(500), QT(500)
      DTMAX=STORE*1000.0
      DELTS=DELT*60.0
      VOLIN=0.0
      DO 101 I=1,LAST
  101 VOLIN=VOLIN+GR(I)*DELTS
C
C     .... PRINT 200,GRPK
C
      QOUT=0.0
      QINC=GRPK/50.0
  102 J=0
      VOLMAX=0
      MIKE=LAST
      DO 103 K=1,500
         QT(K)=0
  103 CONTINUE
      VOL=0
```

```
      QOUT=QOUT+QINC
      IF (QOUT) 109,109,104
  104 J=J+1
      AVAIL=GR(J)+VOL/DELTS
      DIFF=AVAIL-QOUT
      IF (DIFF) 105,105,106
  105 QT(J)=AVAIL
      VOL=0
      GO TO 108
  106 QT(J)=QOUT
      VOL=DIFF*DELTS
      IF (VOLMAX.GT.VOL) GO TO 107
      VOLMAX=VOL
  107 IF (VOL.GT.DTMAX) GO TO 102
  108 CONTINUE
C
C     .... PRINT 300,J,QOUT,AVAIL,DIFF,GR(J),QT(J),VOL,VOLMAX,DTMAX
C
      IF (J.LT.LAST) GO TO 104
      IF (VOL.LT.5.0) GO TO 112
      MIKE=MIKE+1
      IF (MIKE.GT.499) GO TO 110
      GR(MIKE)=0.0
      GO TO 104
  109 WRITE (6,114)
  110 QT(500)=0.0
      VOLOUT=0.0
      DO 111 I=1,500
  111 VOLOUT=VOLOUT+QT(I)*DELTS
      WRITE (6,115)
      WRITE (6,116) VOLIN,VOLOUT
      ALOSS=VOLIN-VOLOUT
      WRITE (6,117) ALOSS
  112 GRPK=QOUT
      VOL=VOLMAX
      LAST=MIKE
      DO 113 K=1,LAST
         GR(K)=QT(K)
  113 CONTINUE
C
C     .... PRINT 201,GRPK
C
      RETURN
C
C
  114 FORMAT (33H NO SOLUTION IN SUBROUTINE DETEN )
  115 FORMAT ( 45H***** ARRAY OVERRUN IN SUBROUTINE DETEN *****)
  116 FORMAT ( 39H***** CONTINUITY CHECK : INPUT VOLUME =,F15.1, 11H CUB
     1IC FEET,/,24X, 15HOUTPUT VOLUME =,F15.1, 11H CUBIC FEET,   6H *****
     2)
  117 FORMAT (   6H***** ,F15.1, 33H CUBIC FEET OF RUNOFF LOST IN SUB, 19
     1HROUTINE DETEN *****)
C
      END
      SUBROUTINE PAVENT (PENT,PL,PS,CPA)
C
C     .... PRINT 6,PENT,PL,PS,CPA
C
      Q=CPA/4.0
      XN=0.02
      S=PS/100.0
      R=0.2
      V=(1.486/XN)*R**0.67*S**0.5
      PENT=PL/V/60.0+2.0
      WRITE (6,101) PENT
      RETURN
C
C
  101 FORMAT (19H PAVED ENTRY TIME= ,F6.1,4H MIN)
C
```

```
      END
      SUBROUTINE RHUFF (TRAIN,DURA,DELT,RR,NRI)
      COMMON /KELLY/ IQUAR
      REAL RR(500),PCTT(17),PCTR(17),SR(500)
      INTEGER XRI
      X=-4.
      DO 101 I=1,11
         X=X+4.0
         PCTT(I)=X
  101 CONTINUE
      DO 102 I=12,17
         PCTT(I)=PCTT(I-1)+10.
  102 CONTINUE
      GO TO (103,104,105,106), IQUAR
  103 PCTR(1)=0
      PCTR(2)=9.6
      PCTR(3)=21.0
      PCTR(4)=32.7
      PCTR(5)=43.
      PCTR(6)=51.2
      PCTR(7)=58.3
      PCTR(8)=63.1
      PCTR(9)=67.2
      PCTR(10)=70.6
      PCTR(11)=73.5
      PCTR(12)=79.5
      PCTR(13)=84.2
      PCTR(14)=88.5
      PCTR(15)=92.5
      PCTR(16)=96.3
      PCTR(17)=100.
      GO TO 107
  104 PCTR(1)=0.0
      PCTR(2)=8.
      PCTR(3)=12.
      PCTR(7)=17.
      PCTR(5)=19.
      PCTR(6)=22.
      PCTR(7)=25.
      PCTR(8)=27.
      PCTR(9)=30.
      PCTR(10)=35.
      PCTR(11)=38.
      PCTR(12)=46.
      PCTR(13)=73.
      PCTR(14)=85.
      PCTR(15)=93.
      PCTR(16)=97.
      PCTR(17)=100.
      GO TO 107
  105 PCTR(1)=0.
      PCTR(2)=17.
      PCTR(3)=26.
      PCTR(4)=30.
      PCTR(5)=33.
      PCTR(6)=35.
      PCTR(7)=37.
      PCTR(8)=39.
      PCTR(9)=41.
      PCTR(10)=44.
      PCTR(11)=46.
      PCTR(12)=61.
      PCTR(13)=92.
      PCTR(14)=96.
      PCTR(15)=97.
      PCTR(16)=99.
      PCTR(17)=100.
      GO TO 107
  106 PCTR(1)=0.
      PCTR(2)=5.
```

```
      PCTR(3)=9.
      PCTR(4)=12.
      PCTR(5)=14.
      PCTR(6)=17.
      PCTR(7)=19.
      PCTR(8)=22.
      PCTR(9)=25.
      PCTR(10)=28.
      PCTR(11)=30.
      PCTR(12)=40.
      PCTR(13)=58.
      PCTR(14)=77.
      PCTR(15)=86.
      PCTR(16)=95.
      PCTR(17)=100.
  107 XRI=DURA/DELT+1.1
      SR(1)=0
      X=0
      DO 111 I=2,XRI
         X=X+DELT
         PX=(X/DURA)*100.
         DO 108 J=1,17
            IF (PX-PCTT(J)) 109,110,108
  108    CONTINUE
         GO TO 111
  109    SR(I)=(PCTR(J-1)+(PCTR(J)-PCTR(J-1))/(PCTT(J)-PCTT(J-1))*(PX-PC
     1   TT(J-1)))*TRAIN*.01
         GO TO 111
  110    SR(I)=PCTR(J)*TRAIN*.01
  111 CONTINUE
      JJ=XRI
      NRI=JJ
      RR(1)=0.0
      DO 112 J=2,JJ
         RR(J)=SR(J)-SR(J-1)
  112 CONTINUE
C
C     .... PRINT 9,(RR(J),J=1,JJ)
C
      RETURN
C
      END
      SUBROUTINE LIMITQ (GR,GRPK,LAST,QALOW,DELT,BRAN,REACH,VOL)
      DIMENSION GR(500), QT(500)
      DELTS=DELT*60.0
      VOLIN=0.0
      DO 101 I=1,LAST
  101 VOLIN=VOLIN+GR(I)*DELTS
C
C     .... PRINT 200,(GR(J),J=1,LAST)
C
      QOUT=QALOW
      J=0
      VOLMAX=0
      MIKE=LAST
      DO 102 K=1,500
         QT(K)=0.0
  102 CONTINUE
      VOL=0.0
  103 J=J+1
      AVAIL=GR(J)+VOL/DELTS
      DIFF=AVAIL-QOUT
      IF (DIFF) 104,104,105
  104 QT(J)=AVAIL
      VOL=0
      GO TO 106
  105 QT(J)=QOUT
      VOL=DIFF*DELTS
      IF (VOLMAX.GT.VOL) GO TO 106
      VOLMAX=VOL
```

```
  106 CONTINUE
      IF (J.LT.LAST) GO TO 103
      IF (VOL.LT.5.0) GO TO 109
      MIKE=MIKE+1
      IF (MIKE.GT.499) GO TO 107
      GR(MIKE)=0.0
      GO TO 103
  107 VOLOUT=0.0
      QT(500)=0.0
      DO 108 I=1,500
  108 VOLOUT=VOLOUT+QT(I)*DELTS
      WRITE (6,111)
      WRITE (6,112) VOLIN,VOLOUT
      ALOSS=VOLIN-VOLOUT
      WRITE (6,113) ALOSS
  109 GRPK=QOUT
      VOL=VOLMAX
      LAST=MIKE
      DO 110 K=1,LAST
         GR(K)=QT(K)
  110 CONTINUE
C
C     .... PRINT 201,(GR(J),J=1,LAST)
C
      RETURN
C
  111 FORMAT ( 46H***** ARRAY OVERRUN IN SUBROUTINE LIMITQ *****)
  112 FORMAT ( 39H***** CONTINUITY CHECK : INPUT VOLUME =,F15.1, 11H CUB
     1IC FEET,/,24X, 15HOUTPUT VOLUME =,F15.1, 11H CUBIC FEET,   6H *****
     2)
  113 FORMAT (  6H***** ,F15.1, 33H CUBIC FEET OF RUNOFF LOST IN SUB, 20
     1HROUTINE LIMITQ *****)
C
      END
      SUBROUTINE ROUTE (GR,IB,DELT,RUFF,SO,DIAM,LENG,LAST,SURCH,Q,VOL,HY
     1D,ISECT,HR,WR,SS,GRPK,BRAN,REACH,IDXRTE,IRUN,RUFFN,TDIAM,QFB)
      DIMENSION GR(500), Q(7,501), TMPGR(5000), TMPQ(5000), TEMP(500)
      COMMON CAPAC,FLAREA,VELOC,AO(51),QO(51),LSTSCT
      COMMON /AOQO/ AOP(51),QOP(51)
      REAL LENG,LS
      INTEGER HYD
      LOGICAL SRCHRG,DEBUG,DESIGN,SIMUL,BEGIN
      SLOP=SO/100.
      IEND=LAST
      LSTSCT=500
      SRCHRG=.FALSE.
      SIMUL=SRCHRG
      DESIGN=SIMUL
      DEBUG=DESIGN
      IF (HYD.NE.0) DEBUG=.TRUE.
      IF (IRUN.EQ.1) DESIGN=.TRUE.
      IF (IRUN.EQ.2) SIMUL=.TRUE.
      IF (DESIGN) ROUGH=RUFFN
      IF (DESIGN) CAPAC=QFB
      IF (DESIGN) FLAREA=3.141592654*TDIAM*TDIAM/(144.*4)
      IF (DESIGN) VELOC=CAPAC/FLAREA
      IF (DESIGN) ISECT=1
      IF (SIMUL) ROUGH=RUFF
      IF (SIMUL) DIA=DIAM
      DO 101 I=1,500
         TEMP(I)=0.0
         TMPQ(I)=0.0
         TMPGR(I)=TMPQ(I)
  101 Q(IB,I)=0.0
      VOLIN=0.0
      XVOL=0.0
      DO 102 I=1,IEND
  102 VOLIN=VOLIN+GR(I)*DELT*60.
      IF (DEBUG.AND.ISECT.NE.3) WRITE (6,135) GRPK
      IF (DEBUG) WRITE (6,136) VOLIN
```

```
      IF (VOLIN.EQ.0.0.OR.GRPK.EQ.0.0) RETURN
      IF (DESIGN) GO TO 106
      GO TO (105,103,108), ISECT
  103 FLAREA=HR*WR
      HRAD=FLAREA/(2.*HR+2.*WR)
      VELOC=1.486*HRAD**(2./3.)*SQRT(SLOP)/ROUGH
      CAPAC=VELOC*FLAREA
      I=1
      DEPTH=0.0
      QO(1)=DEPTH
      AO(1)=QO(1)
  104 I=I+1
      IF (I.EQ.51) GO TO 113
      DEPTH=0.02*HR+DEPTH
      AO(I)=AO(I-1)+0.02
      QO(I)=AO(I)**(5./3.)*((2.*HR+2.*WR)/(2.*DEPTH+WR))**(2./3.)
      GO TO 104
  105 DIA=DIA/12.
      FLAREA=DIA**2*3.141592654/4.0
      VELOC=1.486*(DIA/4.0)**(2./3.)*SQRT(SLOP)/ROUGH
      CAPAC=FLAREA*VELOC
  106 IF (LSTSCT.EQ.1) GO TO 114
      DO 107 I=1,51
         AO(I)=AOP(I)
         QO(I)=QOP(I)
  107 CONTINUE
      GO TO 114
  108 LS=1./SS
      TEMPHR=0.0
  109 TEMPHR=TEMPHR+0.5
C
      HRAD=TEMPHR*(WR+TEMPHR*LS)/(WR+2.*SQRT(1.+LS*LS)*TEMPHR)
      FLAREA=TEMPHR*(WR+TEMPHR*LS)
      CAPAC=(1.486/ROUGH)*FLAREA*HRAD**(2./3.)*SQRT(SLOP)
      VELOC=CAPAC/FLAREA
C
      IF (CAPAC.LT.GRPK) GO TO 109
      DEPTH=0.02*TEMPHR
      I=1
      AO(1)=0.0
      QO(1)=0.0
  110 I=I+1
      AO(I)=AO(I-1)+0.02
      IF (I.EQ.51) GO TO 113
  111 FXN=AO(I)-(DEPTH*(WR+DEPTH*LS))/(TEMPHR*(WR+TEMPHR*LS))
      IF (ABS(FXN).LT.0.05) GO TO 112
      DFXNDD=-(WR+2.*DEPTH*LS)/(TEMPHR*(WR+TEMPHR*LS))
      DEPTH=DEPTH-FXN/DFXNDD
      GO TO 111
  112 PO=(WR+2.*SQRT(1.+LS*LS)*DEPTH)/(WR+2.*SQRT(1.+LS*LS)*TEMPHR)
      QO(I)=AO(I)**(5./3.)/PO**(2./3.)
      GO TO 110
  113 AO(51)=1.
      QO(51)=1.
  114 CONTINUE
      BEGIN=.FALSE.
      DO 115 I=1,500
         IF (BEGIN) GO TO 116
         IF (GR(I).GT.0.01.AND..NOT.BEGIN) ISTART=I
         IF (GR(I).GT.0.01) BEGIN=.TRUE.
  115 CONTINUE
  116 CONTINUE
      IF (DEBUG) WRITE (6,133) (GR(I),I=1,IEND)
      IF (GRPK.GT.CAPAC) SRCHRG=.TRUE.
C
C     .... WRITE(6,3001) SRCHRG
C
      IF (.NOT.SRCHRG) GO TO 119
      SURCH=0.0
      DO 118 I=ISTART,IEND
```

```
          IF (CAPAC-GR(I)) 117,118,118
  117     SURCH=SURCH+(GR(I)-CAPAC)*DELT*60.
  118 CONTINUE
      PVOL=0.0
      CALL LIMITO (GR,GRPK,IEND,CAPAC,DELT,BRAN,REACH,PVOL)
      VOL=VOL+PVOL
  119 CONTINUE
      IF (DEBUG.AND.SRCHRG.AND.ISECT.NE.3) WRITE (6,137) (GR(I),I=1,IEND
     1)
C
C
      TRAVTIM=LENG/(GRPK/CAREA(GRPK))
      IF (TRAVTIM.LE.DELT*60./10.0) GO TO 120
      GO TO 122
  120 WRITE (6,134)
      DO 121 I=1,IEND
  121 Q(IB,I)=GR(I)
      IQEND=IEND
      VOLOUT=VOLIN
      IF (DEBUG) WRITE (6,138) TRAVTIM
      GO TO 132
  122 IF (DEBUG) WRITE (6,138) TRAVTIM
C
      DT=TRAVTIM/60.
C
      IF (TRAVTIM/60.0.LT.DELT) GO TO 123
      GO TO 124
  123 CALL INTERP (DELT,IEND,GR,DT,IENDP,TMPGR,500,5000)
      IEND=IENDP
      GO TO 126
  124 XNDX=(TRAVTIM/60.)/DELT
      NDX=INT(XNDX)
      IF (XNDX-FLOAT(NDX).GE.0.5) NDX=NDX+1
      DO 125 I=1,IEND
  125 TMPGR(I)=GR(I)
      DT=DELT
C
  126 DT=DT*60.0
C
      ISHIFT=1
      IF (TRAVTIM-DELT*60..GE.1E-04) ISHIFT=NDX
      DO 127 I=ISTART,IEND
  127 TMPQ(I+ISHIFT)=TMPGR(I)
      IQEND=IEND+ISHIFT
      IF (IDXRTE.EQ.1) GO TO 130
C
      J=ISTART-1
      IF (TRAVTIM.GE.DELT*60.) J=ISTART+NDX-1
  128 J=J+1
      O2=TMPQ(J+1)
      ITER=1
      CAREA1=CAREA(TMPGR(J))
      CAREA2=CAREA(TMPQ(J))
      CAREA3=CAREA(TMPGR(J+1))
      ALPHA=(DELT*60./DT)
      PREVF=0.0
C
C     IF (IDXRTE.EQ.2) O2=TMPGR(J)+TMPGR(J+1)-TMPQ(J)+(CAREA2-CAREA3)*LE
C    1NG/DT
C
      IF (O2.GT.CAPAC) O2=CAPAC
      IF (O2.LT.0.0) O2=0.0
      TMPQ(J+1)=O2
      IF (J+1.GT.IQEND) GO TO 130
C
C     IF (IDXRTE.EQ.2) GO TO 140
C
      CONST=TMPGR(J)+TMPGR(J+1)-TMPQ(J)+(CAREA1+CAREA2-CAREA3)*LENG/DT
      CONST=CONST
  129 F=CONST-CAREA(O2)*LENG/DT-O2
```

```
C
      IF (PREVF.LT.0.0.AND.F.GT.0.0) ALPHA=ALPHA*2.0
      IF (PREVF.GT.0.0.AND.F.LT.0.0) ALPHA=ALPHA*2.0
      IF (PREVF.LT.0.0.AND.F.LT.0.0) ALPHA=ALPHA*0.666
      IF (PREVF.GT.0.0.AND.F.GT.0.0) ALPHA=ALPHA*0.666
      IF (ABS(F).LE.0.1.AND.O2.GT.GRPK) O2=GRPK
      IF (ABS(F).LE.0.1) TMPQ(J+1)=O2
      IF (ABS(F).LE.0.1) GO TO 128
      IF (ITER.GE.10) WRITE (6,139) F,TMPGR(J),O2
      O2=O2+F/ALPHA
      PREVF=F
      IF (O2.GT.CAPAC) O2=CAPAC
      IF (O2.LT.0.0) O2=0.0
C
C     IF (ABS(O2-CAPAC).LE.1E-04.AND.F.GT.0.0) XVOL=XVOL+F
C
      IF (ABS(O2-CAPAC).LE.1E-04.AND.F.GT.0.0.AND.O2.GT.GRPK) O2=GRPK
      IF (ABS(O2-CAPAC).LE.1E-04.AND.F.GT.0.0) GO TO 128
      IF (O2.EQ.0.0.AND.F.LT.0.0) GO TO 128
      ITER=ITER+1
      IF (ITER.GT.10) WRITE (6,140) ITER
      IF (ITER.GT.25) WRITE (6,141)
      IF (ITER.GT.25) STOP
      GO TO 129
  130 CONTINUE
C
      CALL INTERP (DT/60,IQEND,TMPQ,DELT,IQENDP,TEMP,5000,500)
      IQEND=IQENDP
      VOLOUT=0.0
      DO 131 I=1,IQEND
         Q(IB,I)=TEMP(I)
  131 VOLOUT=VOLOUT+Q(IB,I)*DELT*60.
  132 IF (DEBUG) WRITE (6,142) VOLOUT
      IF (DEBUG) WRITE (6,143) (Q(IB,I),I=1,IQEND)
      IF (DESIGN) CAPAC=0.0
      IF (DESIGN) VELOC=0.0
      LSTSCT=ISECT
      RETURN
C
C
  133 FORMAT (34H DESIGN HYDRO BEFORE CAPACITY LTD ,/,10F8.1,49(/,10F8.1
     1))
  134 FORMAT (49H  TRAVEL TIME SO SMALL THAT ROUTED =DESIGN HYDRO.)
  135 FORMAT (38H PEAK Q BEFORE REACH CAP. REDUCTION = ,F10.1,5H.CFS.)
  136 FORMAT (35H CONTINUITY CHECK--INFLOW VOLUME = ,F15.1)
  137 FORMAT (51H DESIGN HYDRO AFTER REDUCTION FOR REACH CAPACITY    ,/,1
     10F8.1,49(/,10F8.1))
  138 FORMAT (29H TRAVEL TIME FOR THIS REACH =,F10.2,9H SECONDS.)
  139 FORMAT (19H ZEROED CONT FXN = ,F10.2,18H PRESENT INFLOW = ,F10.2,4
     16H PRESENT OUTFLOW FOR WHICH CONTINUITY FAILS = ,F10.2)
  140 FORMAT (75H EXCESSIVE ITERATIONS IN ROUTE FOR IMPLICIT METHOD ITER
     1ATIONS PRESENTLY AT ,I5,1H.)
  141 FORMAT (81H TERMINATION DUE TO EXCESSIVE ITERATIONS OF IMPLICIT ME
     1THOD IN SUBROUTINE ROUTE;;)
  142 FORMAT (36H CONTINUITY CHECK--OUTFLOW VOLUME = ,F15.1)
  143 FORMAT (20H ROUTED DESIGN HYDRO,/,10F8.1,49(/,10F8.1))
C
      END
      FUNCTION CAREA(Q)
      COMMON CAPAC,FLAREA,VELOC,A0(51),Q0(51),LSTSCT
      RATIO=Q/CAPAC
      IF (ABS(RATIO-Q0(51)).LE.1E-04) CAREA=FLAREA
      IF (ABS(RATIO-Q0(51)).LE.1E-04) RETURN
      IF (ABS(RATIO).LE.1E-04) CAREA=0.0
      IF (ABS(RATIO).LE.1E-04) RETURN
      IF (RATIO.LE.Q0(2)) I=2
      IF (RATIO.LE.Q0(2)) GO TO 102
      DO 101 I=2,50
         IF (ABS(RATIO-Q0(I)).LT.ABS(RATIO-Q0(I-1)).AND.ABS(RATIO-Q0(I))
     1   .LT.ABS(RATIO-Q0(I+1))) GO TO 102
```

```
  101 CONTINUE
      IF (ABS(RATIO-Q0(50)).GE.ABS(RATIO-Q0(51))) GO TO 103
      WRITE (6,104) Q
      WRITE (6,105) RATIO
      IF (RATIO.GT.Q0(51)) I=51
      IF (RATIO.GT.Q0(51)) RATIO=1.0
      IF (RATIO.LT.0.0) I=2
      IF (RATIO.LT.0.0) RATIO=0
  102 CONTINUE
      Y1=A0(I-1)
      Y2=A0(I)
      Y3=A0(I+1)
      X1=Q0(I-1)
      X2=Q0(I)
      X3=Q0(I+1)
      A=(Y1-Y3-(Y2-Y3)*(X1-X3)/(X2-X3))/(X1**2-X3**2-(X1-X3)*(X2+X3))
      B=(Y2-Y3)/(X2-X3)-(X2+X3)*A
      C=Y3-X3*(X3*A+B)
      CAREA=(A*RATIO**2+B*RATIO+C)*FLAREA
C
C     .... PRINT *,A,B,C,RATIO,CAREA,I,X1,X2,X3,Y1,Y2,Y3,
C     .... Q0(I+1),A0(I+1)
C
      RETURN
  103 CAREA=FLAREA*(A0(50)+(RATIO-Q0(50))/(Q0(51)-Q0(50))*(A0(51)-A0(50)
     1))
      RETURN
C
C
  104 FORMAT (46H DISCHARGE OUT OF RANGE OF A0 VS Q0 CURVE Q = ,F10.1)
  105 FORMAT (23H DISCHARGE RATIO(Q0) = ,F10.5)
C
      END
      BLOCK DATA AREDIS
C
      COMMON /A0Q0/ A0(51),Q0(51)
      DATA A0/0.0000,.00005,.00042,.00141,.00332,.00645,.01105,.01737,.0
     12562,.03599,.04863,.06366,.08116,.10116,.12366,.14863,.17599,.2056
     22,.23737,.27105,.30645,.34333,.38141,.42042,.46005,.50000,.53995,.
     357958,.61859,.65667,.69354,.72895,.76263,.79438,.82401,.85136,.876
     434,.89884,.91884,.93634,.95137,.96401,.97438,.98263,.98895,.99355,
     5.99667,.99859,.99958,.99995,1.0000/
      DATA Q0/0.0000,.000001,.00002,.00012,.00040,.00104,.00225,.00432,.
     100755,.01231,.01895,.02785,.03939,.05391,.07179,.09306,.11814,.147
     205,.17982,.21639,.25658,.30016,.34675,.39594,.44722,.50000,.55367,
     3.60757,.66102,.71334,.76388,.81202,.85718,.89884,.93658,.97005,.99
     4899,1.02325,1.04278,1.0576,1.06787,1.0738,1.07571,1.07395,1.06898,
     51.06126,1.05131,1.03966,1.02687,1.01347,1.0000/
C
      END
      SUBROUTINE INTERP (DFROM,ENDFROM,ARRFROM,DTO,K,ARRTO,RNGFROM,RNGTO
     1)
      DIMENSION ARRFROM(5000), ARRTO(5000), X(5000)
      INTEGER ENDFROM,RNGTO,RNGFROM
      TOTO=DTO
      VOLIN=0.0
      VOLOUT=0.0
      DO 101 I=1,ENDFROM
  101 VOLIN=VOLIN+ARRFROM(I)*DFROM*60.0
      IF (ENDFROM.EQ.RNGFROM) ARRFROM(ENDFROM)=0.0
      ARRTO(1)=0.0
      K=2
      X(1)=0.0
      IF (ENDFROM.GT.RNGFROM) WRITE (6,110)
      IF (ENDFROM.GT.RNGFROM) WRITE (6,111) 6
      IF (ENDFROM.GT.RNGFROM) STOP
      HMAX=0.0
      DO 102 I=1,ENDFROM
         IF (ARRFROM(I).GT.HMAX) HMAX=ARRFROM(I)
  102 CONTINUE
```

```
      DO 103 I=2,ENDFROM
  103 X(I)=X(I-1)+DFROM
  104 IF (TOTO.LE.DFROM) JP=2
      IF (TOTO.LE.DFROM) GO TO 106
      IF (TOTO-X(ENDFROM).GT.1E-04) GO TO 109
      JKLOW=INT(TOTO/DFROM)
      IF (JKLOW.LE.2) JKLOW=2
      DO 105 JK=JKLOW,ENDFROM
         IF (JK.EQ.ENDFROM) JP=ENDFROM-1
         IF (JK.EQ.ENDFROM) GO TO 106
         IF (ABS(X(JK)-TOTO).LT.ABS(X(JK-1)-TOTO).AND.ABS(X(JK)-TOTO).LT
     1   .ABS(X(JK+1)-TOTO)) JP=JK
         IF (ABS(X(JK)-TOTO).LT.ABS(X(JK-1)-TOTO).AND.ABS(X(JK)-TOTO).LT
     1   .ABS(X(JK+1)-TOTO)) GO TO 106
  105 CONTINUE
  106 Y1=ARRFROM(JP-1)
      Y2=ARRFROM(JP)
      Y3=0
      IF (JP.NE.ENDFROM) Y3=ARRFROM(JP+1)
      X1=X(JP)-DFROM
      X2=X(JP)
      X3=X(JP)+DFROM
      A=(Y1-Y3-(Y2-Y3)*(X1-X3)/(X2-X3))/(X1**2-X3**2-(X1-X3)*(X2+X3))
      B=(Y2-Y3)/(X2-X3)-(X2+X3)*A
      C=Y3-X3*(X3*A+B)
      ARRTO(K)=A*TOTO**2+B*TOTO+C
      IF (ARRTO(K).LT.0.0) ARRTO(K)=0.0
      IF (ARRTO(K).GT.HMAX) ARRTO(K)=HMAX
      K=K+1
C
C     IF (K.GT.RNGTO) WRITE (6,1010)
C     IF (K.GT.RNGTO) WRITE (6,1015)
C     IF (K.GT.RNGTO) STOP
C
      IF (K.GT.RNGTO) GO TO 107
      TOTO=TOTO+DTO
      GO TO 104
  107 K=K-1
      ARRTO(K)=0.0
      DO 108 I=1,RNGTO
  108 VOLOUT=VOLOUT+ARRTO(I)*DTO*60.0
      ALOSS=VOLIN-VOLOUT
      WRITE (6,112)
      WRITE (6,113) VOLIN,VOLOUT
      WRITE (6,114) ALOSS
  109 ARRTO(K)=0.0
      RETURN
C
  110 FORMAT (1H0,50H INTERPOLATED-FROM ARRAY OUT OF RANGE IN INTERP...)
  111 FORMAT (1H0,37H      EXECUTION TERMINATED INTERP.....)
  112 FORMAT ( 47H***** INTERPOLATED-TO ARRAY OUT OF RANGE IN SUB, 20HRO
     1UTINE INTERP *****)
  113 FORMAT ( 39H***** CONTINUITY CHECK : INPUT VOLUME =,F15.1, 11H CUB
     1IC FEET,/,24X, 15HOUTPUT VOLUME =,F15.1, 11H CUBIC FEET,   6H *****
     2)
  114 FORMAT (   6H***** ,F15.1, 33H CUBIC FEET OF RUNOFF LOST IN SUB, 20
     1HROUTINE INTERP *****)
C
      END
      SUBROUTINE EXIST (ISECT,DIAM,RUFF,SLP,HR,WR,SS,ECAP,EVEL)
      REAL LS
      ECAP=0.0
      EVEL=0.0
      IF (ISECT.EQ.0) RETURN
      GO TO (101,102,103), ISECT
  101 IF (DIAM.EQ.0.0) RETURN
      ECAP=0.0081*DIAM*DIAM/RUFF*(DIAM/48.0)**.6667*(SLP/100.0)**0.5
      EVEL=ECAP/(DIAM*DIAM*3.14159/576.0)
      RETURN
  102 IF (HR.EQ.0.0.OR.WR.EQ.0.0) RETURN
```

```
      ECAP=1.486*HR*WR/RUFF*(HR*WR/(2.*HR+2.*WR))**.6667*(SLP/100.)**0.5
      EVEL=ECAP/(HR*WR)
      RETURN
  103 IF (WR.EQ.0.0.OR.HR.EQ.0.0.OR.SS.EQ.0.0.OR.SS.GT.9999.0) RETURN
      LS=1./SS
      BFR=HR*(WR+HR*LS)/(WR+2.*SQRT(1.+LS*LS)*HR)
      BFA=HR*(WR+HR*LS)
      ECAP=(1.486/RUFF)*BFA*BFR**0.6667*(SLP/100.0)**0.5
      EVEL=ECAP/BFA
      RETURN
C
      END
      SUBROUTINE MHHK
      REAL NBAR
      REAL L(50)
      DIMENSION EMAX(50), EDEL(50), J(50), VMIN(50), VMAX(50), DIAM(16),
     1 F(50,50), ESTAR(50,50), EHATS(50,50), STAR(50), HATS(50), EMIN(50
     2), DMT(50,50), DIT(50)
      COMMON /HAN/ Q(50)
      COMMON /HAN5/ GRND(50),EI,EIHAT
      COMMON /HAN2/ PC(16),NHP,FL,AL,OCL,FC,AC,OCC,HPC(13),CEC(7),HPV(20
     1),HPT(20),CE(20)
      COMMON /HAN3/ VOL
      INTEGER XI(50,9),U(50),S(50)
      READ (5,134) NCAP
C
C     ******************************************************************
C     DYNAMIC PROGRAM FOR DESIGN OF  SEWER SYSTEM
C     NBAR = MANNINGS 'N'
C      NCAP = TOTAL NO. OF MANHOLES IN THE SEWER  SYSTEM
C      XI(N,I)= SET OF MANHOLES(I=1,NCAP) EMPTYING  INTO MANHOLE 'N'
C     IN LINES 5 TO 15(FOR EACH MANHOLE(N=1,NCAP) THE SET XI(N,I)IS READ
C     IF NO U/S MANHOLE  EXISTS,N IS PUT IN THE SET S,'(S(N)=1)'
C     IF ANY U/S MANHOLE EXISTS, N IS PUT INTHE SET U,'(U(N)=1)'
C     LINES 19 TO 20 READ THE DATA CONCERNING EACH MANHOLE
C     EMAX(N)= MAX ALLOWABLE ELEVATION FOR MANHOLE N (FT)
C     EMIN(N)= MIN ALLOWABLE ELEVATION FOR MANHOLE N (FT)
C     EDEL(N)=DELTA INTERVAL BETWEEN POTENTIAL ELEVS FOR MANHOLE N(FT)
C     Q(N)=TOTAL VOLUME OF FLOW EXITING FROM MANHOLE N(CFS)
C     L(N) = DISTANCE FROM MANHOLE N TO NEXT DOWNSTREAM MANHOLE(FT)
C     VMIN(N)= MIN ALLOWABLE FLOW VEL BELOW MANHOLE  N (FT/SEC)
C     VMAX(N)= MAX ALLOWABLE FLOW VEL BELOW MANHOLE  N (FT/SEC)
C     GRND(N)= GROUND ELEVATION AT MANHOLE N (FT)
C     LINE 22 READS THE AVAIL ABLE PIPE DIAMETERS (IN),  DIAM(K)
C     IA = INDEX OF ELEMENTS OF SET XI FOR MANHOLE N
C     D= SMALLEST AVAILABLE PIPE DIAMETER NOT LESS THAN DIAM
C     THE FOLLOWING (COMMENT) DATA CARDS  GIVE  THE FORM OF DATA INPUT
C     CARD NO 1.  VALUE OF  NCAP FORMAT  I2
C     CARD NO 2.VALUE OF NBAR  FORMAT  F10.6
C      CARD NO 3 TO (NCAP+3) (XI(N,I),I=1,8) FORMAT 8I10 (NCAP CARDS)
C     CARD NO(NCAP+4)TO(2NCAP+4)VALUES OF EMAX(N),EMIN(N),EDEL(N) Q(N),
C     CARD NO (2NCAP+5) VALUE OF (DIAM(K),K=1,8) FORMAT 8F10.4
C     ******************************************************************
C
      READ (5,135) NBAR
C
C     PRINT 6,(Q(I),I=1,50)
C     6 FORMAT(X,10F10.3)
C
      DO 102 N=1,NCAP
         READ (5,136) (XI(N,I),I=1,8)
         IF (XI(N,1).EQ.0) GO TO 101
         S(N)=0
         U(N)=1
         GO TO 102
  101    U(N)=0
         S(N)=1
  102 CONTINUE
C
C      DO 301 N=1,NCAP
```

```
C      PRINT,/,'S(N)=',S(N),'U(N)=',U(N)
C      301     CONTINUE
C
       DO 103 N=1,NCAP
          READ (5,137) EMAX(N),EMIN(N),EDEL(N),L(N),UMIN(N),UMAX(N),GRND(
      1   N)
C
C      WRITE(6,9010) EMAX(N), EMIN(N), EDEL(N), L(N), UMIN(N), UMAX(N),
C      &GRND(N)
C
  103 CONTINUE
      READ (5,138) (DIAM(K),K=1,16)
      CALL CINPUT
  104 DO 105 N=1,NCAP
         IF (U(N).EQ.1) GO TO 106
  105 CONTINUE
      GO TO 119
  106 DO 108 N=1,NCAP
         IF (U(N).EQ.0) GO TO 108
         DO 107 I=1,8
            IF (XI(N,I).EQ.0) GO TO 107
            IF (S(XI(N,I)).EQ.0) GO TO 108
  107    CONTINUE
         GO TO 109
  108 CONTINUE
      PRINT , /,'ERRORATLINE40MHH'
      STOP
  109 EN=EMAX(N)
      IEN=1.+(EN-EMIN(N))/EDEL(N)
  110 F(N,IEN)=0.
      IA=1
  111 I=XI(N,IA)
      YI=9999999999.0
      EI=EMAX(I)
      IEI=1.+(EI-EMIN(I))/EDEL(I)
  112 EIHAT=EMAX(N)
  113 SLOPE=(EI-EIHAT)/L(I)
      IF (SLOPE.LT..000000000001) GO TO 117
      DIA=(2.15*Q(I)*NBAR/SQRT(SLOPE))**(3./8.)
      DIAIN=DIA*12.0
      VEL=Q(I)/(3.14*((DIA/2.)**2.))
C
C     PRINT 121, VEL
C     121 FORMAT(X,20(F5.1,X))
C
      IF ((VEL.GT.UMAX(I)).OR.(VEL.LT.UMIN(I))) GO TO 117
      D=1000.
      DO 114 K=1,16
         IF (DIAIN.GT.DIAM(K)) GO TO 114
         IF (D.LE.DIAM(K)) GO TO 114
         D=DIAM(K)
  114 CONTINUE
      IF (D.LT.999.) GO TO 115
      PRINT , /,'NOPIPEISLARGEENOUGH'
      STOP
  115 CALL COSTMOD (L,NCAP,COST,I,D,N)
      G=COST+F(I,IEI)
      IF (G.LT.YI) GO TO 116
      GO TO 117
  116 YI=G
      ESTAR(I,IEN)=EI
      EHATS(I,IEN)=EIHAT
      DMT(I,IEN)=D
  117 EIHAT=EIHAT-EDEL(N)
      IF (EIHAT.GE.EN) GO TO 113
      EI=EI-EDEL(I)
      IEI=1.+(EI-EMIN(I))/EDEL(I)
      IF (EI.GE.EMIN(I)) GO TO 112
      F(N,IEN)=F(N,IEN)+YI
      IA=IA+1
```

```
      IF (XI(N,IA).EQ.0) GO TO 118
      GO TO 111
  118 EN=EN-EDEL(N)
      IEN=1.+(EN-EMIN(N))/EDEL(N)
      IF (EN.GE.EMIN(N)) GO TO 110
      U(N)=0
      S(N)=1
      NLAST=N
      GO TO 104
  119 N=NLAST
      EN=EMAX(N)
      IEN=1.+(EN-EMIN(N))/EDEL(N)
      FNCAP=999999999.0
  120 G=259.4+56.4*(GRND(N)-EN)
      IF (ESTAR(N,IEN).GT.EHATS(N,IEN)) GO TO 121
      GO TO 122
  121 Y=9999999999.0
  122 Y=F(N,IEN)+G
      F(N,IEN)=Y
      IF (Y.GE.FNCAP) GO TO 123
      FNCAP=Y
      PRINT 139, FNCAP
      STAR(N)=EN
  123 EN=EN-EDEL(N)
      IEN=1.+(EN-EMIN(N))/EDEL(N)
      IF (EN.GE.EMIN(N)) GO TO 120
      PRINT 140, FNCAP
C
C     PRINT ,/,'OPTIMAL ELEVATION FOR MANHOLE',N,'=',STAR(N)
C
      U(N)=1
      S(N)=0
  124 DO 125 N=1,NCAP
          IF (S(N).EQ.1) GO TO 126
  125 CONTINUE
      GO TO 131
      STOP
  126 DO 129 I=1,NCAP
          IF (S(I).EQ.0) GO TO 129
          DO 128 N=1,NCAP
              IF (U(N).EQ.0) GO TO 128
              DO 127 M=1,8
                  IF (XI(N,M).EQ.I) GO TO 130
  127         CONTINUE
  128     CONTINUE
  129 CONTINUE
      PRINT , /,'ERRORATLINE620MHH'
      STOP
  130 IEN=1.+(STAR(N)-EMIN(N))/EDEL(N)
      STAR(I)=ESTAR(I,IEN)
      HATS(I)=EHATS(I,IEN)
      DIT(I)=DMT(I,IEN)
C
C     PRINT ,/,/,'OPTIMAL ELEVATION OF MANHOLE',I,'=',STAR(I)
C     PRINT,/,'OPTIMAL ELEVATION AT  WHICH THE PIPE FROM',I,'TO',N,
C    *'ENTERS',N,'=',HATS(I)
C
      J(I)=N
C
C      PRINT,/,/,'DIA OF PIPE FROM ',I,'TO',N,'=',DIT(I)
C
      S(I)=0
      U(I)=1
      GO TO 124
  131 CONTINUE
C
C     641  PRINT,/,'MANNINGGS N=',NBAR,/,/,/
C
      PRINT 141
      PRINT , 'NEMAXEMINEDELQLUMINUMAXGRNDXI',/,/
```

```
      DO 132 N=1,NCAP
         WRITE (6,142) N,EMAX(N),EMIN(N),EDEL(N),Q(N),L(N),VMIN(N),VMAX(
     1   N),GRND(N),XI(N,1),XI(N,2),XI(N,3),XI(N,4),XI(N,5),XI(N,6),XI(N
     2   ,7),XI(N,8)
  132 CONTINUE
      PRINT 143
      PRINT 144
      PRINT , /,'MANHOLEOPTELPIPETOMNHLENTERINGELDIA'
      DO 133 I=1,NCAP
         WRITE (6,145) I,STAR(I),J(I),HATS(I),DIT(I)
  133 CONTINUE
C
C     PRINT,/,/,/,/,'AVAILABLE PIPE DIAMETERS ARE'
C     DO 710  M=1,16
C     PRINT,DIAM(M)
C     710  CONTINUE
C
      PRINT , 'STOPPINGATLINE710'
      RETURN
C
  134 FORMAT (I2)
  135 FORMAT (F10.6)
  136 FORMAT (8I10)
  137 FORMAT (7F10.3)
  138 FORMAT (8F10.0)
  139 FORMAT (X, 27HTRIAL TOTAL COST OF SYSTEM=,F20.0)
  140 FORMAT (X, 42HMINIMUM TOTAL COST OF ENTIRE SEWER SYSTEM=,F20.0)
  141 FORMAT (1H1)
  142 FORMAT (  1H0,I5,5X,F8.1,2X,F8.1,2X,F7.1,3X,F7.1,3X,F8.1,2X,F7.1,3
     1X,F7.1,3X,F8.1,2X,8I3)
  143 FORMAT (1H1)
  144 FORMAT ( 47H***************OPTIMAL VALUES*****************)
  145 FORMAT (5X,I6,3X,F8.3,4X,I6,8X,F8.3,4X,F8.3)
C
      END
      SUBROUTINE CINPUT
      COMMON /HAN2/ PC(16),NHP,FL,AL,OCL,FC,AC,OCC,HPC(13),CEC(7),HPV(20
     1),HPT(20),CE(20)
      COMMON /HAN3/ VOL
      READ (5,102) (PC(I),I=1,16)
      READ (5,103) NHP
      J=NHP
      IF (J.EQ.0) GO TO 101
      HPV(1)=VOL**0.666667*5./9.
      READ (5,104) (HPT(I),I=1,J)
      CE(1)=VOL/27.
  101 CONTINUE
      FL=(VOL)**0.333*4.
      AL=(FL/4.)**2/43560.
      READ (5,104) FC,AC
      READ (5,102) (HPC(I),I=1,13)
      READ (5,102) (CEC(I),I=1,7)
      RETURN
C
  102 FORMAT (8F10.2)
  103 FORMAT (40I2)
  104 FORMAT (10F8.0)
C
      END
      SUBROUTINE COSTMOD (PL,NP,PVC,IJ,D,N)
      DIMENSION PL(20), CC(4)
      COMMON /HAN5/ GRND(50),EI,EIHAT
      COMMON /HAN2/ PC(16),NHP,FL,AL,OCL,FC,AC,OCC,HPC(13),CEC(7),HPV(20
     1),HPT(20),CE(20)
      COMMON /HAN3/ VOL
      CC(2)=0.0
C
C     CONTRUCTION COSTS
C     COST 1 IS PIPE
C     COST 2 IS HOLDING POND COST
```

```
C      COST 3 IS OPEN CHANNEL COST
C      COST 4 IS THE TOTAL COST
C
       DEPTHIN=GRND(IJ)-EI
       IF (D.LE.12.0) D1=PC(1)
       IF (D.EQ.15) D1=PC(2)
       IF (D.EQ.18) D1=PC(3)
       IF (D.EQ.21) D1=PC(4)
       IF (D.EQ.24) D1=PC(5)
       IF (D.EQ.27) D1=PC(6)
       IF (D.EQ.30) D1=PC(7)
       IF (D.EQ.33) D1=PC(8)
       IF (D.EQ.36) D1=PC(9)
       IF (D.EQ.39.OR.D.EQ.42) D1=PC(10)
       IF (D.EQ.45.OR.D.EQ.48) D1=PC(11)
       IF (D.EQ.51.OR.D.EQ.54) D1=PC(12)
       IF (D.EQ.57.OR.D.EQ.60) D1=PC(13)
       IF (D.EQ.63.OR.D.EQ.66) D1=PC(14)
       IF (D.EQ.69.OR.D.EQ.72) D1=PC(15)
       IF (D.EQ.75.OR.D.EQ.78) D1=PC(16)
       IF ((DEPTHIN.LE.20.0).AND.(D.LE.36.0)) GO TO 101
       IF ((DEPTHIN.GT.20.0).AND.(D.LE.36.0)) GO TO 102
       IF (D.GT.36.0) GO TO 103
   101 CC(1)=PL(IJ)*D1+PL(IJ)*(1.93*D+1.688*DEPTHIN-12.6)+259.4+56.4*DEPT
      1HIN
       GO TO 104
   102 CC(1)=PL(IJ)*D1+PL(IJ)*(0.696*D+2.14*DEPTHIN+0.559*D*DEPTHIN-13.56
      1)+259.4+56.4*DEPTHIN
       GO TO 104
   103 CC(1)=PL(IJ)*D1+PL(IJ)*(3.638*D+5.17*DEPTHIN-111.72)+259.4+56.4*DE
      1PTHIN
   104 J=NHP
       IF (J.EQ.0) GO TO 106
       DO 105 I6=1,J
          Z1=HPV(I6)
          Z2=HPT(I6)
          IF (Z1.GE.25000.AND.Z2.EQ.10.0) X1=HPC(13)
          IF (Z1.LT.25000.AND.Z1.GE.10000.0.AND.Z2.EQ.10.0) X1=HPC(12)
          IF (Z1.LT.10000.0.AND.Z1.GE.0.0.AND.Z2.EQ.10.0) X1=HPC(11)
          IF (Z1.GE.25000.0.AND.Z2.EQ.9.0) X1=HPC(10)
          IF (Z1.LT.25000.0.AND.Z1.GE.10000.0.AND.Z2.EQ.9.0) X1=HPC(9)
          IF (Z1.LT.10000.0.AND.Z1.GE.5000.0.AND.Z2.EQ.9.0) X1=HPC(8)
          IF (Z1.LT.5000.0.AND.Z1.GE.0.0.AND.Z2.EQ.9.0) X1=HPC(7)
          IF (Z1.GE.25000.0.AND.Z2.EQ.8.0) X1=HPC(6)
          IF (Z1.LT.25000.0.AND.Z1.GE.10000.0.AND.Z2.EQ.8.0) X1=HPC(5)
          IF (Z1.LT.10000.0.AND.Z1.GE.5000.0.AND.Z2.EQ.8.0) X1=HPC(4)
          IF (Z1.LT.5000.0.AND.Z1.GE.0.0.AND.Z2.EQ.8.0) X1=HPC(3)
          IF (Z1.GE.25000.0.AND.Z2.EQ.7.0) X1=HPC(2)
          IF (Z1.LT.25000.0.AND.Z1.GE.0.0.AND.Z2.EQ.7.0) X1=HPC(1)
          CC(2)=CC(2)+X1*HPV(I6)+AL*AC
          Z3=CE(I6)
          IF (Z3.GE.100000.0) X2=CEC(7)
          IF (Z3.LT.100000.0.AND.Z3.GE.50000.0) X2=CEC(6)
          IF (Z3.LT.50000.0.AND.Z3.GE.25000.0) X2=CEC(5)
          IF (Z3.LT.25000.0.AND.Z3.GE.10000.0) X2=CEC(4)
          IF (Z3.LT.10000.0.AND.Z3.GE.5000.0) X2=CEC(3)
          IF (Z3.LT.5000.0.AND.Z3.GE.2000.0) X2=CEC(2)
          IF (Z3.LT.2000.0.AND.Z3.GE.0.0) X2=CEC(1)
          CC(2)=CC(2)+X2*CE(I6)+FL*FC
   105 CONTINUE
   106 CC(4)=CC(1)+CC(2)
       PVC=CC(4)
       RETURN
C
       END
```

INPUT TO ILLUDAS

(Upper Ross-Ade Watershed)

(see Fig. 2.4 and Table 2.2)

```
  UPPER ROSS-ADE WATERSHED
     1         1                          1
   29.12     0.182     0.154       2      12     0.013
                         5.0       1     20.0       5.      1.12        2
1 0            240  3.4                                       1
1 0     1.5     27      13       300  3.0     60      300  3.0
1 1            260  3.4                                       1
1 1     2.3     22      17       400  3.0     61      200  3.0
2 0            250  4.8                                       1
2 0     3.0     20      20       430  3.0     60      350  3.0
2 1            280  4.0                                       1
2 1     1.1     18      18       280  3.0     64      280  3.0
     2  1
1 2            240  2.0                                       1
1 2     1.2     17      17       250  3.0     66      110  3.0
1 3            536  2.8                                 10    1
1 3     9.0     20      16       500  3.0     64      110  3.0
1 4            860  1.5                                   END 1
1 4    11.0     20      17       400  3.0     63      200  3.0
```

INPUT TO DYNAMIC PROGRAMMING

(Upper Ross-Ade Watershed)

8								
0.013								
	0							
	1							
	0							
	3							
	4	2						
	5							
	6							
	7							
	704.000	684.000	1.000	240.000	2.000	8.000	710.000	
	704.000	684.000	1.000	260.000	2.000	8.000	710.000	
	704.000	684.000	1.000	250.000	2.000	8.000	710.000	
	704.000	684.000	1.000	280.000	2.000	8.000	710.000	
	704.000	684.000	1.000	240.000	2.000	8.000	710.000	
	704.000	684.000	1.000	536.000	2.000	8.000	710.000	
	704.000	684.000	1.000	860.000	2.000	8.000	710.000	
	704.000	684.000	1.000	0.000	2.000	8.000	710.000	
	12	15	18	21	24	27	30	33
	36	42	48	54	60	66	72	78
	5.25	7.00	8.70	11.35	14.45	17.45	20.70	26.00
	31.10	41.10	49.95	61.40	78.80	96.40	116.80	130.00
1								
	7							
	9.0	10000						
	12.98	11.85	16.67	15.46	15.30	11.72	18.46	13.80
	14.94	14.22	24.56	18.35	15.60			
	4.10	5.75	3.34	2.75	3.30	2.15	1.51	

OUTPUT FROM ILLUDAS (Upper Ross-Ade Watershed)

```
  ILLUDAS ** ILLINOIS STATE WATER SURVEY **
ILLUDAS UPDATED  OCT,  1979  WITH COMMERCIAL PIPE SIZING
TIME SHIFT ROUTING ACTIVATED.


 UPPER ROSS-ADE WATERSHED

RAINFALL PATTERN
     0    .666    .224    .123    .106

   RUN NUMBER   BASIN AREA   TIME INCREMENT   SOIL GROUP
                  ACRES         MINUTES       1234=ABCD

       1          29.1           5.0              2

    TOTAL RAIN   FREQUENCY   DURATION    AMC   PAVED ABS.   GRASS ABS.
      INCHES       YEARS      MINUTES           INCHES        INCHES

       1.12          5         20.0       2       .18           .15


   B   R   LENG  SLP    N   HT   BW  V/H   DIA  CAPAC  VEL   DESIGN   INLET   DETENTION   STORAGE
            FT   PCT        FT   FT        INS   CFS   FPS   Q-CFS    Q-CFS   CUBIC FT    REQUESTED


 ****  BEGIN BRANCH   1. REACH     0 ****


PAVED ENTRY TIME=    3.1 MIN
    ACCUM CONTRIBUTING AREAS   CPA=     .4,   SPA=     .2,   CGA=     .9
PAVED AREA HYDROGRAPH        1.0          0
      0     2.4     1.1     .6      .5
GRASSED ENTRY TIME=   35.3 MIN
GRASSED AREA HYDROGRAPH
      0     .4      .4      .4      .4      .4      .4      .4      .0
PEAK Q BEFORE REACH CAP. REDUCTION =       2.7.CFS.
CONTINUITY CHECK--INFLOW VOLUME =        2184.1
```

```
DESIGN HYDRO BEFORE CAPACITY LTD
     0     2.7      1.5      1.0       .9        .4        .4        .4        .0
TRAVEL TIME FOR THIS REACH =      30.08 SECONDS.
CONTINUITY CHECK--OUTFLOW VOLUME =             2230.6
ROUTED DESIGN HYDRO
     0     2.6      1.6      1.0       .9        .4        .4        .4        .1        0

     1.    0  240.  3.40  .013   -0    -0    -0    -0        0       0        0                        0              0

                 REQUIRED PIPE =              12.    6.56  8.36      2.74      2.74          0



 ****  BEGIN BRANCH    1. REACH     1. ****


PAVED ENTRY TIME=     3.5 MIN
    ACCUM CONTRIBUTING AREAS    CPA=       .9,   SPA=      .6,   CGA=     2.3
PAVED AREA HYDROGRAPH          1.0        1.0
     0     2.9      1.4       .7       .6
GRASSED ENTRY TIME=    30.9 MIN
GRASSED AREA HYDROGRAPH
     0      .8       .8       .8       .8        .8        .8        .1
UPSTREAM ROUTED PLUS SURFACE HYDROGRAPH.
     0     6.4      3.7      2.6      2.4       1.2       1.2        .5
PEAK Q BEFORE REACH CAP. REDUCTION =          6.4.CFS.
CONTINUITY CHECK--INFLOW VOLUME =            5398.1
DESIGN HYDRO BEFORE CAPACITY LTD
     0     6.4      3.7      2.6      2.4       1.2       1.2        .5
 TRAVEL TIME SO SMALL THAT ROUTED =DESIGN HYDRO.
TRAVEL TIME FOR THIS REACH =      27.31 SECONDS.
CONTINUITY CHECK--OUTFLOW VOLUME =             5398.1
ROUTED DESIGN HYDRO
     0     6.4      3.7      2.6      2.4       1.2       1.2        .5
     1.    1.  260.  3.40  .013   -0    -0    -0    -0        0       0        0                        0              0

                 REQUIRED PIPE =              12.    6.56  8.36      6.39      3.74          0



****  BEGIN BRANCH   2. REACH     0 ****
```

```
PAVED ENTRY TIME=    3.6 MIN
    ACCUM CONTRIBUTING AREAS   CPA=    1.5,   SPA=    1.2,   CGA=    4.1
PAVED AREA HYDROGRAPH       2.0         0
      0     3.5     1.6     .9      .8
GRASSED ENTRY TIME=   37.8 MIN
GRASSED AREA HYDROGRAPH
      0     .9      .9      .9      .9      .9      .9      .9      .5
PEAK Q BEFORE REACH CAP. REDUCTION =        4.4.CFS.
CONTINUITY CHECK--INFLOW VOLUME =        4162.9
DESIGN HYDRO BEFORE CAPACITY LTD
      0     4.4     2.6     1.8     1.7     .9      .9      .9      .5
 TRAVEL TIME SO SMALL THAT ROUTED =DESIGN HYDRO.
TRAVEL TIME FOR THIS REACH =     24.41 SECONDS.
CONTINUITY CHECK--OUTFLOW VOLUME =         4162.9
ROUTED DESIGN HYDRO
      0     4.4     2.6     1.8     1.7     .9      .9      .9      .5
     2.   0  250. 4.80  .013   -0    -0    -0    -0       0     0      0                    0            0

                 REQUIRED PIPE =            12.    7.80  9.93     4.43     4.43             0


 ****  BEGIN BRANCH   2. REACH    1. ****


PAVED ENTRY TIME=    3.1 MIN
    ACCUM CONTRIBUTING AREAS   CPA=    1.7,   SPA=    1.4,   CGA=    4.8
PAVED AREA HYDROGRAPH       2.0         1.0
      0     1.2     .5      .3      .3
GRASSED ENTRY TIME=   34.3 MIN
GRASSED AREA HYDROGRAPH
      0     .4      .4      .4      .4      .4      .4      .3
UPSTREAM ROUTED PLUS SURFACE HYDROGRAPH.
      0     5.9     3.4     2.5     2.3     1.3     1.3     1.3
PEAK Q BEFORE REACH CAP. REDUCTION =        5.9.CFS.
CONTINUITY CHECK--INFLOW VOLUME =        5419.5
DESIGN HYDRO BEFORE CAPACITY LTD
      0     5.9     3.4     2.5     2.3     1.3     1.3     1.3
 TRAVEL TIME SO SMALL THAT ROUTED =DESIGN HYDRO.
TRAVEL TIME FOR THIS REACH =     27.60 SECONDS.
CONTINUITY CHECK--OUTFLOW VOLUME =         5419.5
ROUTED DESIGN HYDRO
      0     5.9     3.4     2.5     2.3     1.3     1.3     1.3
     2.   1.  280. 4.00  .013   -0    -0    -0    -0       0     0      0                    0            0
```

```
          REQUIRED PIPE =            12.    7.12  9.06     5.94    1.51          0


 ****  BEGIN BRANCH    1. REACH    2. ****


PAVED ENTRY TIME=    3.0 MIN
    ACCUM CONTRIBUTING AREAS   CPA=    1.9,   SPA=    1.6,   CGA=    5.6
PAVED AREA HYDROGRAPH        1.0       2.0
     0    1.2     .5     .3      .3
GRASSED ENTRY TIME=   24.5 MIN
GRASSED AREA HYDROGRAPH
     0     .5     .5     .5      .5      .5
UPSTREAM ROUTED PLUS SURFACE HYDROGRAPH.
     0    14.1    8.3    5.9     5.5     3.0
PEAK Q BEFORE REACH CAP. REDUCTION =       14.1.CFS.
CONTINUITY CHECK--INFLOW VOLUME =        11017.0
DESIGN HYDRO BEFORE CAPACITY LTD
     0    14.1    8.3    5.9     5.5     3.0
 TRAVEL TIME SO SMALL THAT ROUTED =DESIGN HYDRO.
TRAVEL TIME FOR THIS REACH =     25.12 SECONDS.
CONTINUITY CHECK--OUTFLOW VOLUME =       11017.0
ROUTED DESIGN HYDRO
     0    14.1    8.3    5.9     5.5     3.0
    1.  2.  240. 2.00  .013   -0   -0   -0   -0     0    0      0              0         0
          REQUIRED PIPE =            18.   14.84  8.40    14.06    1.73          0


 ****  BEGIN BRANCH    1. REACH    3. ****


PAVED ENTRY TIME=    3.9 MIN
    ACCUM CONTRIBUTING AREAS   CPA=    3.7,   SPA=    3.0,   CGA=   11.4
PAVED AREA HYDROGRAPH        1.0       3.0
     0    10.5    4.8    2.7     2.3
GRASSED ENTRY TIME=   25.4 MIN
GRASSED AREA HYDROGRAPH
     0     3.7    3.7    3.7     3.7     3.7      .3
```

```
UPSTREAM ROUTED PLUS SURFACE HYDROGRAPH.
     0    28.2    16.8    12.3    11.5     6.7     .3
PEAK Q BEFORE REACH CAP. REDUCTION =       9.0.CFS.
CONTINUITY CHECK--INFLOW VOLUME =       22781.7
DESIGN HYDRO BEFORE CAPACITY LTD
     0     9.0     9.0     9.0     9.0     9.0     9.0     9.0     9.0     3.6

TRAVEL TIME FOR THIS REACH =     53.55 SECONDS.
CONTINUITY CHECK--OUTFLOW VOLUME =       22916.5
ROUTED DESIGN HYDRO
     0     8.1     9.0     9.0     9.0     9.0     9.0     9.0     9.0     5.0
     0
    1.  3.  536. 2.80  .013   -0   -0   -0   -0     0     0   28.25   14.19          0

              REQUIRED PIPE =          18.   17.56  9.94     9.04                9819.6       10000.


 ****  BEGIN BRANCH   1. REACH    4. ****


PAVED ENTRY TIME=    3.5 MIN
    ACCUM CONTRIBUTING AREAS   CPA=    5.9,   SPA=    4.9,   CGA=   18.3
PAVED AREA HYDROGRAPH        1.0      4.0
     0    12.8     5.9     3.3     2.8
GRASSED ENTRY TIME=   30.9 MIN
GRASSED AREA HYDROGRAPH
     0     3.9     3.9     3.9     3.9     3.9     3.9      .7
UPSTREAM ROUTED PLUS SURFACE HYDROGRAPH.
     0    24.8    18.8    16.2    15.7    12.9    12.9     9.7
PEAK Q BEFORE REACH CAP. REDUCTION =      24.8.CFS.
CONTINUITY CHECK--INFLOW VOLUME =       33309.2
DESIGN HYDRO BEFORE CAPACITY LTD
     0    24.8    18.8    16.2    15.7    12.9    12.9     9.7
TRAVEL TIME FOR THIS REACH =     86.29 SECONDS.
CONTINUITY CHECK--OUTFLOW VOLUME =       33410.8
ROUTED DESIGN HYDRO
     0    20.8    20.3    16.7    16.1    13.4    13.2    11.0      0
    1.  4.  860. 1.50  .013   -0   -0   -0   -0     0     0     0                 0          0

              REQUIRED PIPE =          24.   27.68  8.81    24.79   16.66          0
```

OUTPUT FROM DYNAMIC PROGRAMMING

(Upper Ross-Ade Watershed)

```
 OUTFALL HYDROGRAPH IN CFS, ACCUMULATED RUNOFF IN CU FT=      33411.
      0     20.8     20.3     16.7     16.1     13.4     13.2     11.0
 TRIAL TOTAL COST OF SYSTEM=             175349.
 TRIAL TOTAL COST OF SYSTEM=             175161.
 TRIAL TOTAL COST OF SYSTEM=             174033.
 TRIAL TOTAL COST OF SYSTEM=             167809.
 TRIAL TOTAL COST OF SYSTEM=             164626.
 TRIAL TOTAL COST OF SYSTEM=             160165.
 TRIAL TOTAL COST OF SYSTEM=             156982.
 TRIAL TOTAL COST OF SYSTEM=             155624.
 MINIMUM TOTAL COST OF ENTIRE SEWER SYSTEM=              155624.
1
```

	N	EMAX	EMIN	EDEL	Q	L	UMIN	UMAX	GRND	XI
0	1	704.0	684.0	1.0	2.7	240.0	2.0	8.0	710.0	0 -0 -0 -0 -0 -0 -0 -0
0	2	704.0	684.0	1.0	6.4	260.0	2.0	8.0	710.0	1 -0 -0 -0 -0 -0 -0 -0
0	3	704.0	684.0	1.0	4.4	250.0	2.0	8.0	710.0	0 -0 -0 -0 -0 -0 -0 -0
0	4	704.0	684.0	1.0	5.9	280.0	2.0	8.0	710.0	3 -0 -0 -0 -0 -0 -0 -0
0	5	704.0	684.0	1.0	14.1	240.0	2.0	8.0	710.0	4 2 -0 -0 -0 -0 -0 -0
0	6	704.0	684.0	1.0	9.0	536.0	2.0	8.0	710.0	5 -0 -0 -0 -0 -0 -0 -0
0	7	704.0	684.0	1.0	24.8	860.0	2.0	8.0	710.0	6 -0 -0 -0 -0 -0 -0 -0
0	8	704.0	684.0	1.0	0	0	2.0	8.0	710.0	7 -0 -0 -0 -0 -0 -0 -0
1										

**************OPTIMAL VALUES*****************

MANHOLE	OPT EL	PIPE TO MNHL	ENTERING EL	DIA
1	704.000	2	703.000	15.000
2	703.000	5	702.000	18.000
3	704.000	4	703.000	18.000
4	703.000	5	702.000	18.000
5	702.000	6	701.000	24.000
6	701.000	7	697.000	18.000
7	697.000	8	691.000	27.000
8	691.000	0	0	0

STOPPING AT LINE 710

1

THE JOB IS FINISHED

APPENDIX B

RECENT DEVELOPMENTS IN SEWER SYSTEM DESIGN

Many researchers are working on the problems of drainage system design. The purpose of this appendix is to report on the latest published efforts of other researchers in this area which became available after the work described in this report was completed.

The principal advancement by other researchers is reported in "Advanced Methodology for Storm Sewer Design, Phase II" by Wenzel, Yen, and Tang (1979). They have constructed a large computer model that should facilitate the analysis of drainage problems by combining ILLUDAS and several versions of a DDDP optimization. The optimization model considers detention storage as a component of the drainage system. Two versions of the optimization model are available. The first version will find the minimum cost sewer system when the locations and maximum outflow rates for all detention storages are specified as inputs to the model. The second version attempts to identify, within the model, the best locations and sizes of detention storages as well as the best sewer system design. In order to construct this second version, the assumption that reducing the peak flow in one pipe will not reduce the peak flows in downstream pipes had to be made. Clearly, this assumption is not valid. Consequently, the drainage system design specified by the second version of the model will not be optimal in the sense of minimizing the total cost.

The work described in this report is based on a model similar to that developed by Wenzel et al. (1979) although the model discussed in the present study was constructed independently at Purdue University. The most significant differences between the Purdue model and the model by Wenzel et al. (1979) are:

(1) a dynamic program was used instead of a discrete differential dynamic program, and

(2) no attempt was made in the present study to formulate an optimization model that considered detention storage in a suboptimal manner.

The computer model developed by Wenzel et al. (1979) could be used instead of the computer model described in this report. All of the sensitivity analysis reported here could be undertaken with either model. However, it is important to note that Wenzel et al. (1979) have not discussed the results of sensitivity analysis such as those in the present study. Consequently, although the model structures are similar in certain respects the results discussed in the two studies are quite different.

The effects of uncertainties in physical inputs to the model and in the economic variables used to estimate costs on the selection of drainage design can be studied by using different approaches. In the present study, these effects are investigated by using a sensitivity analysis approach to the problem. By following this approach the variations in costs of alternative designs can be studied and the most suitable design can be selected.

Another approach to investigate the effects of uncertainties in the physical inputs and in the economic varibles is to assume probability distributions which characterize the physical and economic uncertainties and the risks of failure. By using appropriate assumptions about these distributions and about the dependencies between the distributions, the probabilities of failure or risks can be estimated. This approach has been used by Tang et al. (1975, 1976) and Mays (1979), for example.

Both of these approaches have their merits. As a sufficient number of case studies is not available in the literature to enable us to prefer one approach to the other, both of them should be considered as valid and useful.

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